

Total Emission Analysis of Sewerage Systems and Wastewater Treatment Plants

Andrew G. Jack (B.Eng.)

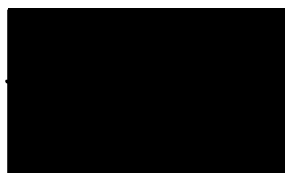
A Thesis submitted in partial fulfilment of the requirements of the University of Abertay Dundee for the degree of Doctor of Philosophy.

This research programme was carried out in collaboration with the North of Scotland Water Authority.

September 1999

I certify that this thesis is the true and accurate version of the thesis approved by the examiners.

Signed



(Director of Studies)

Date *10th October 1999*

CONTENTS

Contents	i
List of Figures	x
List of Tables	xv
Notation	xix
Abbreviations	xxii
Acknowledgements	xxiv
Abstract	xxvi
 Chapter One:- Introduction, Objectives and Thesis Overview	 1
1.1 Introduction	1
1.2 Thesis Overview	3
 Chapter Two:- Literature Review Introduction: Separate and Combined Systems	 5
2.1 Separate and Combined Sewerage Systems	5
2.2 Separate versus Combined Sewerage Systems	5
2.21 First Foul Flush	6
2.22 Gully Pots	7
2.23 Benefit of Combined Systems	7
2.3 Conclusions	7
 Chapter Three:- Wastewater Treatment	 9
3.1 Introduction	9
3.2 Physical Processes	9
3.21 Screens	9
3.22 Grit Removal	9
3.23 Flow Equalisation	10
3.24 Sedimentation	10
3.241 Primary Sedimentation	10
3.242 Final Sedimentation Tank	11
3.3 Biological Unit Processes	12
3.31 Activated Sludge	12
3.4 The Effects of Transient Loading On Unit Processes	14
3.41 Primary Clarifier under Combined Loading	15
3.42 Secondary Clarifier Under Combined Loading	15
3.43 Biological Reactor under Combined Loading	16
3.5 Summary	17

CONTENTS

Chapter Four:- The Receiving Watercourse	19
4.1 Introduction	19
4.2 Integration	19
4.3 Acute Pollution	20
4.4 Accumulative Pollution	21
4.5 Conclusions	22
Chapter Five:- Interaction Between Sewerage System & WTPs & Literature Review Summary	23
5.1 Introduction	23
5.2 Sewerage systems, Storage and WTP Performance	23
5.3 Total Emission Study	24
5.4 Dilution Ratio	25
5.5 WTP Control	25
5.6 Summary	26
5.7 Sewerage System and WTP Interaction Conclusions	26
5.8 Overall Literature Review Summary	27
5.9 Previous Research and Shortfalls	28
Chapter Six:- Catchment Description	32
6.1 Perth Sewerage System	32
6.2 North Muirton	32
6.21 Hillyland	34
6.22 Craigie	34
6.23 Rannoch	34
6.24 Bridgend	35
6.25 City Centre	35
6.26 Moncrieffe	35
6.3 General System Characteristics	36
6.4 System Deficiencies	36
6.41 Bridgend	36
6.42 North Muirton	37
6.43 Perth Centre	37
6.5 Wastewater Treatment Plant	38

CONTENTS

Chapter Seven:- Sewer Flow and Quality Modelling Overview (WALLRUS & MOSQUITO - Theory)	40
7.1 Introduction	40
7.2 WALLRUS	40
7.3 MOSQUITO (Theory)	41
7.31 Sediment Transport Modelling	41
7.32 Pollutant Origins and Modelling	42
7.33 Sediment Layers	42
7.4 Perth MOSQUITO Model Construction – Dry Weather Flows and Quality	43
7.41 Data Collection	43
7.42 Land Use Identification	43
7.5 Dry Weather Flow Verification	45
7.51 DWQ Verification - TSS	46
7.52 DWQ Verification – COD	46
7.53 DWQ Verification – Ammonia	47
7.6 Summary	47
7.7 Default Values and Site Specific Data	47
Chapter Eight:- MOSQUITO Model Construction:- Storm Flows and Quality	49
8.1 Introduction	49
8.2 Sediment Washoff and Gully Pot Influence	49
8.3 Simplified Model	50
8.4 In-Sewer Processes	51
8.5 Pipe Sediment Data Collection	51
8.6 Summary	52
Chapter Nine:- MOSQUITO Sensitivity Testing	53
9.1 Introduction	53
9.2 Sediment Pollutant Characteristics Sensitivity Test	53
9.3 Summary	54
Chapter Ten:- Hydroworks–PM /Hydroworks-QM	55
10.1 Introduction	55

CONTENTS

Chapter Ten (Cont.)

10.2	Hydroworks-QM:- Background	55
10.3	Differences and Similarities (MOSQUITO & FLUPOL)	55
10.4	Surface Washoff Models (MOSQUITO & FLUPOL)	56
10.5	Sediment Transport Models (MOSQUITO & FLUPOL)	56
10.6	Hydroworks-QM	57
10.7	Hydroworks-QM Washoff Model	57
10.8	Initialisation	58
10.9	Conversion from MOSQUITO to Hydroworks-QM (DWF)	59
10.10	Model Verification – Dry Weather Quality	60
10.11	Comparison of Accuracy –QM & MOSQUITO (DWQ)	61
10.111	Infiltration Modelling	62
10.112	Infiltration Modelling Procedure	62
10.12	Identification of Infiltration for the Perth System	62
10.13	-QM Infiltration Modelling Analysis	64
10.14	Discussion	67
10.15	Ammonia Modelling	67
10.16	Model Construction and Verification – Storm Flows	68
10.17	Conclusions (Hydraulic Model)	72
10.18	Quality Model Construction & Verification – Storms	73

Chapter Eleven:- BOD Modelling 74

11.1	Introduction	74
11.2	BOD – Dry Weather	74
11.3	Storm Verification (BOD)	75
11.31	Rainfall Event 24/8/95	75
11.32	Rainfall Event 29/8/95	76
11.33	Rainfall Event 31/5/95	76
11.4	BOD Modelling Conclusions	77

Chapter Twelve:- Hydroworks-QM Sensitivity Testing 79

12.1	Introduction	79
12.2	Sensitivity Test Strategy	79
12.21	Sensitivity Test 1A	79
12.22	Sensitivity Test 1B	80
12.23	Sensitivity Test 2	80
12.24	Sensitivity Test 3	80

CONTENTS

Chapter Twelve (Cont.)

12.3	Sensitivity Tests:- Discussion and Results	81
12.31	Test 1A (Initialisation Period for Quality Calculations)	81
12.4	Test 1B – Background, Discussion and Results	82
12.5	Test 2 – Infiltration Modelling and Sensitivity of Storm Model Outputs	83
12.6	Test 3 – Sediment Characteristics Sensitivity	84
12.61	Sediment Diameter Adjustment – Background, Discussion and Results	84
12.62	Diameter Test Results	84
12.63	Storm Sensitivity to Diameter Adjustment	85
12.64	Specific Gravity Calibration	85
12.65	Storm Sensitivity to Specific Gravity Adjustments	86
12.7	Sensitivity Test Conclusions - Summary	86
12.8	BOD Sensitivity	87
12.9	Infiltration Modelling Conclusions	87
12.10	Hydroworks –QM – Overall Conclusions	88
12.11	The Inadequate Modelling of TSS	89
12.12	Summary/Conclusions	90
12.121	Development of a Suitable Deposited Sediment Bed Model	90
12.122	Utilisation of MOUSETRAP	92
12.123	Non Deterministic Modelling	92
12.124	Simplistic Modelling Approach	93

Chapter Thirteen:- Waste Water Treatment Plant Modelling 94

13.1	Introduction	94
13.2	Sleepless Inch Waste Water Treatment Plant	94
13.3	Data Collection	94
13.4	Model Calibration	95
13.5	Storm Modelling	97
13.51	Primary Tanks (Storms)	97
13.52	Activated Sludge (Storms)	98
13.6	STOAT Model Summary and Conclusions	99

Chapter Fourteen:- WTP Modelling – GPS-X (Theory) 102

14.1	Introduction	102
14.2	Data Requirements	102
14.3	GPS-X Libraries	102

CONTENTS

Chapter Fourteen (Cont.)

14.31	Calculations	103
14.4	Unit Process Models	105
14.41	Clarifier Models	105
14.42	Modelling Flow Distribution in One Dimensional Settling Tanks	106
14.43	Two Dimensional Settler Model	107
14.44	Biological Models	107
14.5	Model Construction – DWF	108
14.51	Primary Settlement Tank Modelling	108
14.52	Activated Sludge and Final Clarifier Modelling	109
14.6	Model Verification - Storms	110
14.7	Storm Modelling Summary	113
14.8	Comparison of STOAT and GPS-X	113
14.81	Primary Tank Summary	114
14.82	Activated Sludge Tanks	114
14.9	Storm Tank Calibrations	115
14.10	WTP Modelling:- Overall Summary	117

Chapter Fifteen:- WTP Sensitivity Testing 119

15.1	Introduction	119
15.2	Sensitivity Test No.1	119
15.21	Dry Weather Testing	119
15.22	Wet Weather Testing	121
15.23	Test Procedure	122
15.24	Wet Weather Test Results	122
15.3	Sensitivity Test No.2	126
15.31	Additional Assumptions	128
15.32	Test Procedure	128
15.33	Test Results	129
15.4	Conclusions for Sensitivity Test No.2	131
15.5	Sensitivity Test No.3	131
15.51	Conclusion	134
15.52	Summary of TSS/BOD Analysis for Test No.3 (at 1000l/s)	136
15.53	Storm Tanks	137
15.54	Sensitivity of WTP and Storm Tanks with Decreasing Flow Rate	139
15.5	Conclusion for Sensitivity Test No.3	142
15.6	Development of Hypothetical Influent Profile	142
15.7	Development of Hypothetical CSO Spill Pollutant Data	145
15.8	Summary and Conclusions	150

CONTENTS

Chapter Sixteen:- Method Development	152
16.1 Introduction	152
16.2 Analysis Period	152
16.3 Continuous versus Discrete Simulation	155
16.4 Summary	155
16.5 Maximum Storage Volume	156
16.6 Rainfall Profiles	157
16.6.1 Worst Case Scenarios	158
16.7 Definition of Wettest Period	158
16.8 Method Summary	162
 Chapter Seventeen:- Application of Proposed Method to Perth - Rainfall Generation	 163
17.1 Introduction	163
17.2 STORMPAC	163
17.3 Perth	164
 Chapter Eighteen:- Largest Possible Storage Volume Assessment for Perth	 166
18.1 Introduction	166
18.2 General Procedure	166
18.3 CSO Storage & Flood Storage Protection	168
18.4 Perth Case Study	169
18.5 Flood Analysis	170
18.6 Location of Storage to Prevent Flooding	170
18.7 Overflow Performance Evaluation	171
18.8 Summary	177
 Chapter Nineteen:- Total Emission Analysis Period	 178
19.1 Introduction	178
19.2 Worst Case Rainfall Profiles	178
19.3 SIMPOL	180
19.3.1 Calibration	181
19.4 SIMPOL Analysis (Jan '70)	183
19.5 TEAP Assessment	184
19.5.1 Results	186
19.6 Periods of Continued Disruption	186

CONTENTS

Chapter Nineteen (Cont.)

19.7	Sludge Blanket Breakthrough Analysis	187
19.8	Summary	193
19.9	WTP Disruptive Periods of Ammonia Analysis	194
19.10	Determination of Disruptive Periods	196
19.11	TEAP Summary	200

Chapter Twenty:- Total Emission Analysis (TEA) for Perth

20.1	Introduction	204
20.2	Total Emissions - BOD	204
20.3	WTP Performance	205
20.4	Summary and Conclusions (BOD)	210
20.5	Total Emissions - Ammonia	212
20.6	WTP Performance (Ammonia)	213
20.7	Overall Total Emission Analysis Summary – BOD and Ammonia	219

Chapter Twenty One:- Conclusions

21.1	Introduction	222
21.2	Modelling Conclusions	222
21.3	Appropriate Computational Period	225
21.4	Representative Rainfall	226
21.5	Storage and WTP Performance	227
21.6	Best Storage Volume	229
21.7	Two or Three DWF	232

Chapter Twenty Two:- Suggestions for Further Research

22.1	Introduction	234
22.2	Sewer Flow Quality Modelling	234
22.3	Total Emission Analysis (Acute Pollution)	235
22.4	Cost Benefit (Storage/BMPs)	236

References

CONTENTS

Appendices

Appendix A	Plates	A
Appendix B	Hydroworks - QM Quality Model Construction and Verification – Storms	B
Appendix C	WTP Model Comparison:- STOAT (Prototype), STOAT (Commercial) and GPS-X	C
Appendix D	Sewer Flow Quality Modelling - Supporting Graphs	D
Appendix E	Waste Water Treatment Plant Modelling -Supporting Graphs	E
Appendix F	Biological Treatment - Two or Three Times DWF?	F
Appendix G	Publications	G

LIST OF FIGURES

3.1	Transport Zones in Secondary Settlement Tanks	11
3.2	Schematic Diagram of a Denitrifying WTP	14
3.3	Influent Soluble Substrate (Combined Loading)	16
5.1	Total Emissions versus Storage Volume	24
6.1	Drainage Sub-Catchments in Perth	33
6.2	Schematic Diagram of the Perth WTP	39
9.1	Rainfall and Flow Profiles	54
9.2	TSS and BOD (Data sets A and B)	54
10.1	Infiltration Assessment of the Perth Drainage System	63
10.2	-QM Infiltration Comparison at System Outfall - Flow	65
10.3	-QM Infiltration Comparison at System Outfall – Ammonia	65
10.4	-QM Infiltration Comparison at System Outfall – TSS	66
10.5	-QM Infiltration Comparison at System Outfall – COD	66
10.6a	Rainfall Profile for 24/5/95 (Burghmuir)	68
10.6b	Rainfall Profile for 24/5/95 (Perth Grammar)	68
10.6c	Rainfall Profile for 24/5/95 (Murray Royal)	69
10.6d	Modelled and Observed Flow – Bridgend (24/5/95)	69
10.6e	Modelled and Observed Velocity – Bridgend (24/5/95)	69
10.6f	Modelled and Observed Flow – Craigie (24/5/95)	70
10.6g	Modelled and Observed Velocity – Craigie (24/5/95)	70
10.7a	Rainfall Profile for 31/5/95 (Burghmuir)	70
10.7b	Rainfall Profile for 31/5/95 (Perth Grammar)	71
10.7c	Rainfall Profile for 31/5/95 (Murray Royal)	71

LIST OF FIGURES

10.7d	Modelled and Observed Velocity – Bridgend (31/5/95)	71
10.7e	Modelled and Observed Flow – Bridgend (31/5/95)	71
10.7f	Modelled and Observed Flow – Craigie (31/5/95)	72
10.7g	Modelled and Observed Velocity – Craigie (31/5/95)	72
13.1	TSS in Activated Sludge Effluent (STOAT – Storm)	98
13.2	BOD in Activated Sludge Effluent (STOAT – Storm)	98
13.3	Ammonia in Activated Sludge Effluent (STOAT – Storm)	99
14.1	Total Nitrogen Calculation	104
14.2	COD Calculation	105
14.3	TSS in Activated Sludge Effluent (GPS-X - Storm)	112
14.4	BOD in Activated Sludge Effluent (GPS-X - Storm)	112
14.5	Ammonia in Activated Sludge Effluent (GPS-X - Storm)	113
14.6	TSS in Activated Sludge Effluent (STOAT & GPS-X - Storm)	115
14.7	BOD in Activated Sludge Effluent (STOAT & GPS-X - Storm)	115
15.1	WTP DWF Influent Profile	120
15.2	WTP DWQ Influent Profile	120
15.3	WTP BOD Effluent Profiles for Test Scenarios	120
15.4	Hypothetical Rainfall Profiles	121
15.5	Influent Flow Data for WTP Model (Storm)	121
15.6	Influent Quality Data for WTP Model (Storm)	122
15.7	WTP Effluent Profiles for Input Interval Analysis Tests	123
15.8	Flush Sensitivity Analysis – Input Hydrograph	128

LIST OF FIGURES

15.9	Flush Sensitivity Analysis at 600l/s – WTP Effluent TSS	129
15.10	Flush Sensitivity Analysis at 600l/s – WTP Effluent BOD	130
15.11	Flush Sensitivity Analysis at 400l/s – WTP Effluent TSS	130
15.12	Flush Sensitivity Analysis at 400l/s – WTP Effluent BOD	131
15.13	WTP Influent Flow Profile – “Dilution Analysis”	133
15.14	Dilution Analysis at 1000l/s – WTP Effluent (TSS)	134
15.15	Dilution Analysis at 1000l/s – WTP Effluent (BOD)	135
15.16	Dilution Analysis at 1000l/s – Storm Tank Effluent (TSS)	137
15.17	Dilution Analysis at 1000l/s – Storm Tank Effluent (BOD)	138
15.18	Dilution Analysis at 600l/s – WTP Effluent (TSS)	139
15.19	Dilution Analysis at 600l/s – WTP Effluent (BOD)	139
15.20	Dilution Analysis at 500l/s – WTP Effluent (TSS)	140
15.21	Dilution Analysis at 500l/s – WTP Effluent (BOD)	140
15.22	Dilution Analysis at 400l/s – WTP Effluent (TSS)	141
15.23	Dilution Analysis at 400l/s – WTP Effluent (BOD)	141
16.1	WTP Emissions for TSR Event 3	154
16.2	WTP Emissions for TSR Event 4	154
16.3	WTP Emissions for TSR Event 12	154
16.4	TEAP Method Overview – Flow Chart	161
18.1	Flood Locations - Perth (5yr 30min event)	170
18.2	Largest Possible Storage Volume (Flow Chart)	177
19.1	SIMPOL Representation of the Perth Sewerage System	181
19.2	Influent of 800l/s for 20 days – WTP Effluent – Flow	189
19.3	Influent of 800l/s for 20 days – WTP Effluent – BOD	189

LIST OF FIGURES

19.4	Influent of 800l/s for 20 days - Heterotrophic Biomass in Activated Sludge Tank	190
19.5	Influent of 1000l/s (@10mg/l BOD) for 8 days - WTP Effluent – Flow	192
19.6	Influent of 1000l/s (@10mg/l BOD) for 8 days - WTP Effluent – Flow	192
19.7	Influent of 1000l/s (@10mg/l BOD) for 8 days - Heterotrophic Biomass in Activated Sludge Tank	192
19.8	Bacteria Displacement Analysis - WTP Effluent – Flow	198
19.9	Bacterial Displacement Analysis - WTP Effluent – Ammonia	199
19.10	Bacterial Displacement Analysis – Autotrophic Biomass in Activated Sludge Tank	199
19.11	Method for Determining the Total Emission Analysis Period	203
20.1	CSO Spill Load – BOD (0m ³ and 10,000m ³ Storage)	205
20.2	WTP Effluent - Flow (0m ³ and 10,000m ³ Storage)	207
20.3	WTP Effluent - BOD (0m ³ and 10,000m ³ Storage)	207
20.4	Heterotrophic Bacteria in Reactor (0m ³ and 10,000m ³ Storage)	207
20.5	WTP Effluent Load – BOD (0m ³ and 10,000m ³ Storage)	209
20.6	CSO Spill Load - Ammonia (0m ³ and 10,000m ³ Storage)	212
20.7	WTP Effluent - Flow (0m ³ and 10,000m ³ Storage)	214
20.8	WTP Effluent – Ammonia (0m ³ and 10,000m ³ Storage)	215
20.9	Autotrophic Bacteria in Reactor (0m ³ and 10,000m ³ Storage)	215
20.10	WTP Effluent - Flow (10,000m ³ and 20,000m ³ Storage)	215
20.11	Autotrophic Bacteria in Reactor (10,000m ³ and 20,000m ³ Storage)	216
20.12	WTP Effluent - Flow (10,000m ³ and 20,000m ³ Storage)	216

LIST OF FIGURES

20.13	WTP Effluent Load – Ammonia (0m ³ and 10,000m ³ Storage)	217
21.1	Summary of Total Emission Analysis Method	233

LIST OF TABLES

7.1	Land Use Identification and Characteristics	44
7.2	Dry Weather Flow Pollutant Characteristics	45
7.3	Comparison of Site Specific Data and Default Values	48
8.1	Spatial Variability of Pollutants Attached to Sediments in Pipes of Similar Gradient and Land Use in Perth (approx. 1mile sampling interval)	51
8.2	Spatial Variability of Pollutants Attached to Sediments (close proximity – 250m)	52
9.1	Pipe Sediment Characteristics	53
10.1	Land Use Characteristics	59
10.2	Sediment Calibration Parameters	61
10.3	Accuracy of Model Predictions:- MOSQUITO & -QM	61
10.4	Sediment Calibration Parameter Comparison	64
10.5	Infiltration and No Infiltration Model Comparison	64
10.6	Hydraulic Modelling Errors (Volume and Peak Flows)	72
11.1	Comparison of BOD and COD Storm Modelling Errors	77
12.1	Temporal and Spatial Variability of Pollutants Attached to Sediments (Within Single Pipe Length - 48m)	91
13.1	Average and Peak Modelled Errors in Primary Tank	100
13.2	Average and Peak Modelled Errors in Activated Sludge Eff.	100

LIST OF TABLES

14.1	CN Library State Variables	103
14.2	Settling Tank Calibration Parameters	109
14.3	Modelled Errors in Primary Tank Effluent – STOAT & GPS-X	114
14.4	Modelled Errors in Activated Sludge Effluent – STOAT & GPS-X	114
14.5	Storm Tank Data Collection Appraisal	116
15.1	WTP Effluent Load - BOD	123
15.2	WTP Effluent Loads (Varying Input Timestep Intervals)	124
15.3	WTP Effluent Load Analysis (BOD)	124
15.4	WTP Effluent Load Analysis (TSS)	124
15.5	WTP Effluent Load Analysis (Ammonia)	125
15.6	Five Minute, One Hour and Two Hour Input Interval Analysis (TSS)	125
15.7	Five Minute, One Hour and Two Hour Input Interval Analysis (BOD)	126
15.8	TSS Effluent Load Comparison (1 Hour Input Time-step Interval)	127
15.9	BOD Effluent Load Comparison (1 Hour Input Time-step Interval)	127
15.10	WTP Input Data	129
15.11	Hypothetical Influent Pollutant Profile - TSS & BOD	143
15.12	Hypothetical Influent Pollutant Profile - Ammonia	144
15.13	Hypothetical Influent Pollutant Profiles - TSS, BOD & And Ammonia	144
15.14	Rainfall Characteristics	145

LIST OF TABLES

15.15	South Inch Flow and Quality Characteristics	146
15.16	Development of South Inch Diluted Component Hypothetical Data	148
15.17	Bridgend Flow and Quality Characteristics	149
15.18	“Diluted Component” Hypothetical Data	149
15.19	Flush Component Characteristics	150
15.20	Diluted Component Characteristics	150
16.1	TSR Characteristics and Total Emission Analysis Period	153
16.2	Fundamental UPM Acute Pollution Standards (DO)	157
16.3	Fundamental UPM Acute Pollution Standards (Un-ionised Ammonia)	157
16.4	Rain-Day Table	159
17.1	Perth Rainfall Generation Characteristics	164
17.2	Monthly Average Rainfall (MAR) – Published and STORMPAC	164
17.3	Daily Rainfall Totals – Published and STORMPAC	165
18.1	SDD Storage Requirements	167
18.2	South Inch Pumping Station Overflow - Spill Volumes	172
18.3	Willowgate Pumping Station Overflow Analysis	173
18.4	South Inch Pumping Station Overflow Analysis	173
18.5	Friarton Pumping Station Overflow Analysis	174
19.1	Monthly Rainfall Totals (1970 - 1979)	178
19.2	Days of Rainfall in Wettest Month	179

LIST OF TABLES

19.3	SIMPOL Calibrations	181
19.4	Pumping Station Spills	182
19.5	CSO Discharge Loads	184
19.6	WTP Test Data	185
19.7	WTP Disruptive Periods	186
19.8	WTP Disruptive Periods for BOD Analysis	194
19.9	Disruptive Period - Summary Table	197
20.1	CSO Spill Load – BOD (0m ³ and 10,000m ³ Storage)	205
20.2	WTP Effluent - BOD Load	209
20.3	Volumetric Balance	210
20.4	Total Emissions - BOD	210
20.5	CSO Spill - Ammonia	213
20.6	WTP Effluent Load – Ammonia (0m ³ and 10,000m ³ Storage)	217
20.7	Total Emissions – Ammonia	217
20.8	Total Emissions - Ammonia (Critical Period:- hrs 24-116)	218

NOTATION

A_d	Calibration constant for surface type
A_{aw}	Coefficient depending on dimensionless particle diameter
a_e	Calibration constant for surface type
a_i	Calibration constant for surface type
C	Pollutant concentration (mg/l)
C_{max}	Limit sediment concentrations (g/l)
C_{min}	Limit sediment concentrations (g/l)
C_s	Settleable solids concentration (kg/m ³)
C_o	Muskingum flow routing coefficient
C_1	Muskingum flow routing coefficient
C_2	Muskingum flow routing coefficient
C_{aw}	Coefficient depending on dimensionless particle diameter
D	Duration of combined loading resulting from initial event (hrs)
D_u	Duration of rainfall event (hrs)
D_i	Deposition rate of sediment fraction from overland flow (kg/hr)
d_{35}	Particle diameter classification (mm)
E	Trade Effluent flow (l/day)
E_i	Erosion rate of sediment mixture (kg/hr)
E_f	Erosion rate of sediment fraction by overland flow (kg/hr)
F	Pollutant flow (kg/s)
F_{gr}	Dimensionless mobility particle number
F_m	TSS flow (kg/s)
F_o	Substrate concentration (kg/m ³)
G_{gr}	Dimensionless solid flow number
g	Acceleration due to gravity (m/s ²)
h	Coefficient

NOTATION

I	Rainfall intensity (mm/hr)
I_r	In-flow (l/s)
J	Energy line slope
j	Constant
K	Storage constant
K_b	Coefficient
Kf	Erosion/deposition factor related to rainfall intensity
Kpn	Potency factor
KA	Coefficient depending on rainfall intensity
k	Exponential constant for hindered settling
L	Linear reservoir coefficient.
Md	Mass of pollutants present on the ground (kg)
Me	Mass of pollutant dissolved or in suspension (kg)
Mr	Mass of surface deposit pollution (kg)
Ms	Instantaneous mass of suspended solids (kg)
m	Coefficient depending on dimensionless particle diameter
n	Coefficient depending on dimensionless particle diameter
O	Out-flow (l/s)
P	Population Served
p	Exponential constant for settling at low solids concentration
q_t	Total solid flow (kg particles/kg water)
R	WTP Recovery Period (hrs)
R_d	Rainfall Depth (mm)
R_h	Hydraulic radius (m)
rhin	Hindered settling zone parameter (m^3/g)
rflo	Flocculant zone settling parameter (m^3/d)

NOTATION

MLSS	Mixed Liquor Suspended Solids (kg/m^3)
S	Flow section (m^2)
SSVI	Specific Stirred Volume Index (ml/g)
s	Specific gravity of particles
T	Total Emission Analysis Period (hrs)
t	Time (s)
U	Mean flow velocity (m/s)
V	Settling velocity (m/h)
V_o	Maximum possible settling velocity (m/h)
V_{sj}	Settling velocity in layer j (m/d)
V_{so}	Maximum Vesilind settling velocity (m/d)
u^*	Friction velocity (m/s)
We	Effective deposited sediment width (m)
w	Sediment settling velocity (m/s)
X	Dimensionless weighting factor
X_j	Suspended solids concentration in layer j (mg/l)
X_{\min}	Minimum attainable suspended solids concentration (mg/l).
η_{\max}	Efficiency coefficients
η_{\min}	Efficiency coefficients
ρ	Density of water (kg/m^3)
ρ_m	Density of sediment and water (kg/m^3)
ρ_s	Density of sediment (kg/m^3)
τ	Shear stress of the overland flow (N/m^2)
τ_{cd}	Critical shear stress for deposition (N/m^2)
τ_{ce}	Critical shear stress for erosion of sediment fraction (N/m^2).

ABBREVIATIONS

ADWP	Antecedent Dry Weather Period
Ammn	Ammonia
ASP	Activated Sludge Process
ATV	Abwassertechnische Vereinigung
BMP	Best Management Practise
BPEO	Best Practical and Environmental Option
BOD	Biochemical Oxygen Demand
CBOD	Carbonaceous Biochemical Oxygen Demand
CN	Carbon Nitrogen
CNP	Carbon Nitrogen Phosphorous
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
CSTR	Continuously Stirred Tank Reactor
DO	Dissolved Oxygen
DWF	Dry Weather Flow
DWQ	Dry Weather Quality
EMC	Event Mean Concentration
EPSRC	Engineering and Physical Sciences Research Council
EQO	Environmental Quality Objective
EQS	Environmental Quality Standard
FWR	Foundation for Water Research
IAWQ	International Association on Water Quality
IP	Industrial Process
MAR	Monthly Average Rainfall
MLSS	Mixed Liquor Suspended Solids
MOSQUITO	Modelling Sewer Flow Quality Including Tanks and Overflows
NoSWA	North of Scotland Water Authority

ABBREVIATIONS

NRA	National Rivers Authority
PDWF	Peak Dry Weather Flow
RP	Return Period
SDD	Scottish Development Department
SRG	Statistical Rainfall Generator
SSVI	Specific Stirred Volume Index
STKN	Soluble Total Suspended Solids
STOAT	Sewage Treatment Operational Analysis over Time
TEA	Total Emission Analysis
TEAP	Total Emission Analysis Period
TKN	Total Kjeldahl Nitrogen
TSR	Time Series Rainfall
TSS	Total Suspended Solids
UAD	University of Abertay Dundee
UPM	Urban Pollution Management
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WTP	Waste Water Treatment Plant
WRc	Water Research Centre Plc.

Acknowledgements

In a project of this type, there are many people to thank for the assistance and advice freely given and gratefully received:

The Engineering and Physical Science Research Council for the financial support of this project.

The North of Scotland Water Authority for financial support and technical assistance.

Prof. Richard Ashley (Director of Studies)

Sincere thanks are due to Professor Ashley. Many doors were opened simply by Richard's presence, allowing the project to progress at a rate which would otherwise not have been possible. My time of study under Richard has left me only with respect for his intellectual ability and his project management skills.

Dr David Wotherspoon (Project Supervisor)

A debt of gratitude is also owed to Dr Wotherspoon for the consistent advice, criticism and help which he provided throughout this project. A great deal of time and effort were put into the project by David and I would like to express my sincere thanks for all his contributions.

Dr Joseph Akunna (Project Supervisor)

A lot of useful information was passed on by Dr Akunna concerning the biological processes of wastewater treatment. A very good working relationship existed between Dr Akunna and myself which greatly aided the project's overall progress.

Dr Mark Petrie (Colleague)

Although Dr Petrie was never officially involved in this project in a supervisory capacity, his own doctoral work provided a spring board from which this research project commenced. Mark also ungrudgingly gave up a great deal of personal time to discuss his opinions about the strengths and weaknesses of the UPM programme, which this project revolved around. A great deal of thanks are owed to Mark, who is both a good researcher and a good friend.

Alasdair Fraser (Colleague)

The acknowledgement list would not be complete without the mention of Alasdair Fraser. Alasdair assisted in the sewer flow quality storm data collection exercise which was carried out at the onset of the project. Many hours were spent together in a van waiting on rainfall which always threatened but never actually occurred. A lot of funny stories were told and a lot of good memories exist from what could have been the most boring months in history.

Finally, I wish to thank my family for the encouragement and support which they have given me throughout my studies -

I would like this achievement to make you proud.

Andrew Jack

Abstract

The proposed methodology to most effectively manage intermittent combined sewage discharges into urban watercourses in the UK is given in the Urban Pollution Management (UPM) manual. The method is based on the use of detailed computer models of the sewerage system, wastewater treatment plant and receiving watercourse. Solving intermittent discharge problems using UPM, often requires the installation of in-sewer storage tanks. However, recent research from Germany and elsewhere (e.g. Austria and Denmark) has shown that this type of solution may be of little benefit with respect to the total emissions discharged from the entire system, where emissions from both the Combined Sewer Overflows (CSOs) and the Wastewater Treatment Plant (WTP) are considered together. This is because, in certain situations, WTP efficiency can be compromised by the prolonged periods of dilute (low nutrients and substrate) inflows which can result from the draining down of in-sewer storage tanks.

The earlier research in Germany and elsewhere has been concerned with long term total emissions (annual loads) and not the problems specific to individual sites, or the benefits and/or limitations of storage with respect to acute pollution. Thus the principal objective of the research described here has been to substantiate and quantify the total emission problem by means of detailed modelling, via an evaluation of the likely storage volumes which could give rise to total emissions problems for the Perth wastewater system. Following this, a general method has been developed to investigate and resolve total emission problems related to acute pollution effects. As WTP disruption due to flow dilution can last for a prolonged period after even a single rainfall event, computational simulation times need to be long enough to represent the delay in WTP performance returning to normal operating conditions. As long term continuous simulation is usually impractical due to protracted computational times, a method referred to as the Total Emission Analysis Period (TEAP) has been developed. This will define the minimum required computational time and rainfall inputs to be used to ensure that the effect of in-sewer storage on total emissions could be modelled.

Utilising the TEAP method to analyse total emissions it has been concluded that increasing volumes of storage would not be expected to create a total emission

problem with respect to the Biochemical Oxygen Demand (BOD). Consequently, it was concluded that the best storage volume with respect to BOD was the minimum volume which would allow compliance with receiving water quality standards. No direct comparison could be made with the conclusion derived from the German research due to the long term nature of their analysis, however, it would appear from an interpretation of their results, that similar findings were obtained.

With respect to ammonia, it was found that increases in total emissions *can* occur as, ammonia concentrations, unlike BOD, do not increase at the start of a storm due to first foul flush effects. Consequently, any increased emissions from the WTP would not be offset via a reduced CSO spill load. It was also found, however, that increasing volumes of storage would not be expected to exacerbate acute pollution problems within a receiving watercourse and that both large and small storage volumes had the potential to give rise to very similar degrees of WTP disruption. This was due to the way in which different hydraulic loading conditions (caused by the different volumes of storage) affected the bacterial concentrations in the reactor. The conclusion that storage would not provide a significant benefit for ammonia total emissions was supported by the Austrian and Danish research.

Chapter One

Introduction, Objectives and Thesis Overview

1.1 Introduction

Wastewater systems discharge into receiving waters, which receive pollution from both the sewerage system, via Combined Sewer Overflows (CSOs), and the wastewater treatment plant (WTP). The total pollution load discharged is a direct result of a definite interaction between sewers and treatment plants. If a receiving water is to receive discharge from the WTP and the combined sewer overflows, then it is this total load which is of importance (**Durschlag, 1989**). The interaction between the sewerage system and the WTP must therefore be considered if an optimisation of the entire system is to be achieved (**Otterpohl, 1993**). This study is concerned with this interaction, specifically the problems arising from the installation of in-sewer storage of combined flows (to reduce CSO spills) which can cause prolonged periods of dilute (nutrient and substrate limited) flows to enter downstream treatment plants.

The research had the following principal aims:-

1. To develop and evaluate the limitations of computational simulation models using standard software packages, to accurately represent the performance of the sewerage and treatment plant systems for the City of Perth in order to investigate the effects of introducing in-sewer storage to alleviate intermittent discharges on total emissions during storm events.
2. To develop criteria for the selection of the appropriate computational simulation period required to investigate total emissions and to minimise computing time.
3. To develop a method for the selection of a representative rainfall period for total emission analysis investigations.

4. To further develop knowledge about the problems which in-sewer storage of combined flows may cause in terms of disruption to WTP performance.
5. To develop a method to guide engineers through the process of defining the best storage volume (with respect to acute pollution) for any particular catchment under consideration.
6. To assess whether the German standard of 2 times dry weather flow (DWF) for full treatment was preferable in terms of total emissions than the UK standard of 3DWF.

Specific objectives to achieve these aims were:-

1. To review and enhance existing commercial models (MOSQUITO, Hydroworks-QM), by the collection of sewer flow and quality data for the City of Perth.
2. To assess the usefulness of these commercial models for the accurate representation of the changes in BOD and Ammonia which occur in the Perth sewerage system during storm events.
3. To devise appropriate in-sewer storage volumes to alleviate intermittent discharges from the existing CSOs in Perth in accordance with normal UPM procedures.
4. To utilise an existing STOAT model and also develop a new GPS-X model to represent the quality performance in terms of BOD and Ammonia of the Sleepless Inch WTP serving the city of Perth.
5. To produce final modelling tools suitable for the simulation of the total emissions from the Perth wastewater system.
6. To investigate the use of time series rainfall for the simulation of the total emissions from the Perth wastewater systems
7. To investigate the effect of individual and consecutive rainfall events on the disruption of the efficient performance of the Perth WTP.

8. To utilise the computational tools, together with the analysis of disruptive periods to achieve the project aims as defined above.

1.2 Thesis Overview

The reported study has been divided into four main sections to aid readability. These sections are literature review, model building/calibration, method development and method implementation.

The literature review is documented in **chapters two through five**. Chapters two and three are principally introductory chapters which discuss the basic concepts of the sewerage system and wastewater treatment plant. In **chapter two** consideration is given to gully pots, the first foul flush phenomenon and combined and separate sewerage systems. **Chapter three** describes the various unit processes found in suspended growth treatment systems and explains why WTPs are more effective at biological removal and sedimentation processes during dry weather than under storm conditions. **Chapter four** is concerned with the receiving watercourse. The difference between acute and chronic pollution is discussed in this chapter. **Chapter five** draws on the summaries and conclusions from these chapters and highlights where further knowledge was required to develop current total emission understanding. The development of the **aims and objectives (as stated in Chapter 1)** of this research project are summarised in this chapter.

Chapter six links the literature review with the model building/calibration section. This chapter describes the sewerage system and WTP of Perth, from which the computer models used in the study were built. The model building/calibration section (**chapters seven to fifteen**) was detailed prior to the method development section in order to highlight the influence which the modelling problems had on the final method which was developed.

Chapter seven discusses the theory behind sewer flow and sewer flow quality modelling. **Chapters eight to twelve** detail sewer flow quality model calibration for Perth under both dry weather and wet weather conditions. **Chapter twelve** concludes that the available sewer flow quality modelling packages are inadequate for use within this research project due to limitations and problems of the software. The advantages and disadvantages of various alternative measures

are therefore described in the chapter and a recommendation is made as to how sewer flow quality should be represented in this research project.

Chapters thirteen and fourteen are concerned with wastewater treatment plant modelling and describe two alternative competitive software packages (STOAT and GPS-X). Although in general, similar results were obtained it was recommended that GPS-X be used for the research. **Chapter fifteen** details WTP sensitivity tests which were carried out to aid the development of the sewer flow quality approach recommended in **chapter twelve**.

The third section of the thesis is concerned with the detailed development of the total emission analysis method. This section is described in **chapter sixteen**. The novel Total Emission Analysis Period method is introduced in this chapter.

The fourth and final section of the thesis is detailed in **chapters seventeen through twenty**. These chapters work through each stage of the proposed method in detail and highlight how various problems were resolved in the Perth study. The chapters are structured in such a way as to provide worked examples for those who wish to utilise the method to resolve their own particular total emission problems.

Chapter twenty one provides a summary of the entire thesis and highlights the conclusions derived in this project.

Chapter 2

Literature Review Introduction: Separate and Combined Systems

2.1 Separate and Combined Sewerage Systems

A separate sewerage system conveys foul flow to a treatment plant and conveys surface runoff to a receiving watercourse. Two separate sewer pipes are utilised to convey each flow type. At present in the UK, treatment is usually not given to the discharged waste water from surface runoff pipes.

A combined sewerage system utilises only one pipe to convey both foul and storm waters, with both flows being taken to the treatment works. Due to definite practical and financial constraints, the treatment plant should only purify a certain portion of this combined flow (Arsov, 1993). Flows in excess are therefore spilled to a receiving watercourse, normally without receiving explicit treatment, albeit, they may be screened.

During more extreme rainfall events the hydraulic capacity of sewerage networks may be exceeded. This, typically, results in surcharging of sewer lengths and possibly flooding if the surcharging is of sufficient magnitude. However, in order to prevent a sewerage system surcharging and/ or flooding excessively, control structures e.g. combined sewer overflows can operate.

2.2 Separate versus Combined Sewerage Systems

Until recently separate sewerage systems were considered the best approach to receiving water quality management, as only surface waters were spilled to the watercourse. This logic followed the perception that the discharged surface runoff was of relatively clean (chemically and biologically) composition. This

is no longer considered to be the case and was initially identified at least 15 years ago (**Field and Turkeltaub, 1981**). Studies have shown that certain surface runoff can be even more polluting than combined sewer overflow discharges, as strong hydrocarbons, heavy metals, tyre residuals, brake fluids, hazardous spill materials etc. all originating from highways, are discharged directly into the receiving watercourse.

One important aspect of the separate sewerage system, however, is that the foul only pipe has a greater potential to be self cleansing than the corresponding combined sewer system. The reason for this being that combined sewerage systems must convey both foul and storm flows, and so, to prevent excessive hydraulic failure, the pipe sizes are designed using very extreme rainfall. This consequently results in far greater pipe diameters than would be required to convey the typical flows under dry weather conditions. As a result low flow velocities prevail during dry weather days, and thus in turn, sediment deposition is promoted (**Ashley and Crabtree, 1992**). The potential magnitude of the first foul flush (**Ashley et al, 1992a**) phenomenon is strongly related to this, albeit, only in certain sewers (**Stotz & Krauth, 1986**).

2.21 First Foul Flush

During periods of prolonged dry weather a fine layer of organic sediment can build up on top of the consolidated layer of previously deposited sediment in sewers. It is probable that the occurrence of first flushes are related to the nature of the sewer catchment and the sewer system geometry as these properties heavily influence the hydraulic characteristics of the catchment (**Ashley et al, 1992b**). The longer the antecedent dry period, the greater the potential for build up of fine organic material (given the pipe length is governed by frictional forces in preference to gravitational forces). When flows increase, and hence bed shear stresses increase (**Kleijwegt et al, 1990**), the fine organic layer, which travels as a dense cloud close to the bed (**Verbanck, 1994**), may be re suspended and carried into the main flow, as the sediments in this zone are weakly resistant to erosion (**Wotherspoon & Ashley, 1992**). If

the rainfall is of sufficient intensity to cause combined sewer overflow operation, then this re-eroded organic load, which is of course, highly polluted, may be discharged directly into the receiving watercourse (Lindholm, 1976). As combined sewers also convey surface runoff flows, any discharged pollutant load will contain pollutants originating from the catchment surface. Gully pots act to attenuate the surface pollutant contribution, however, they themselves contribute to the in-sewer pollutant concentrations as described below.

2.22 Gully Pots

During prolonged dry weather periods anaerobic conditions prevail within the gully pot, releasing oxygen demanding soluble organics, ammonium and possibly sulphides. In wet weather physical processes dominate - the runoff rapidly displaces the standing liquor into the sewer. This process can represent a significant fraction of the total flow volume and pollutant load contributing to a first foul flush (Butler et al, 1994).

2.23 Benefit of Combined Systems

Although the composite discharged pollutant load from the combined sewerage system is a mixture of both the foul component and that component associated with the surface runoff, it is however, important to remember that, unlike separate sewerage systems, combined sewer overflows do not operate every time it rains.

2.3 Conclusions

It is possible that combined sewerage systems in certain circumstances produce less total pollution than separate systems. However, the question of whether or not combined sewers should be utilised in preference to separate, or vice versa, is complex, with many varying opinions existing. Different countries have different approaches to river quality management and so no consensus exists on which to formulate a definitive answer. Also, due to technical restraints and inadequacies, the answer, to date, could only have been hypothesised. Consequently, the river quality management strategies undertaken by the

different countries have been based on political and planning measures, rather than scientific reasoning. It can be confidently said, however, that no definitive answer can be given 'a priori', without very detailed analysis, and that this answer will not be universal; each sewerage system must be considered independently, and in terms of its individual circumstances.

Chapter Three

Wastewater Treatment

3.1 Introduction

The aim of this chapter is to highlight the stages of treatment commonly encountered at activated sludge treatment plants. Consideration has only been given to the activated sludge process as the research project was solely concerned with this form of biological treatment.

3.2 Physical Processes

The physical unit processes most commonly encountered are

- Screening
- Grit Removal
- Flow equalisation
- Sedimentation

3.21 Screens

The first unit operation encountered in a WTP is screening. Screens are used to protect pumps, valves, pipelines etc. from clogging and damage from rags and large objects. The removal of these objects from the flow also aids the prevention of aesthetic pollution. This process is referred to as pre-treatment.

3.22 Grit Removal

The incomplete removal of grit may cause serious operational problems due to deposits in aeration tanks and the digestors, damage to pumps and clogging of pipes. The grit chamber must therefore be able to demonstrate flow properties

that guarantee a reliable settling of the grit during combined waste water loading conditions. This unit process is also considered to be pre-treatment.

3.23 Flow Equalisation

Flow equalisation can be utilised to overcome operational problems caused by flow rate and quality variation, to improve the performance of the subsequent processes and to reduce the size and cost of the subsequent processes.

The objective of flow equalisation is to dampen the flow rate so that a constant or near constant flow rate is attained. Both dry and wet weather flow variations can be dampened by flow equalisation (Metcalf & Eddy, 1991). This unit process is commonly used only in South Africa as an aid to the nutrient removal processes (Dunn-Flores, 1998).

3.24 Sedimentation

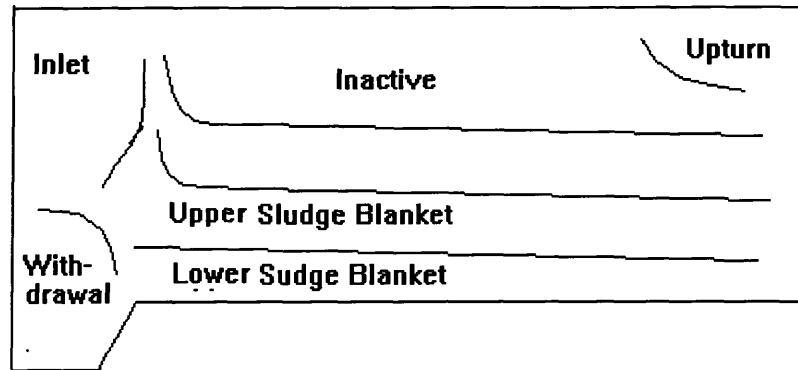
This process is used for grit removal, particulate matter removal in the primary settling unit, biological floc removal from the activated sludge process (ASP (see 3.31)) and chemical floc removal when chemical coagulation processes are being used. The primary purpose is to separate the solids from the liquid, but also to produce sludge with a solids concentration that can be handled and treated easily (Metcalf & Eddy, 1991).

3.241 Primary Sedimentation

The primary sedimentation tank gives first stage or primary treatment to the influent wastewater. The primary tank settles out solids concentrations with an efficiency dependent upon the hydraulic detention time. However the organic content removal is not considered to rise above 35% even with large detention times (> 3hrs) (Harremoes et al, 1993).

3.242 Final Sedimentation Tank

Fig.3.1 Transport Zones In Secondary Settlement Tanks



Transport zones in secondary settling tanks
(Lumley and Balmer, 1990)

Fig. 3.1 shows seven transport zones (Lumley and Balmer, 1990) which can occur in a rectangular final settlement tank. In the influent zone the flow is characterised by high turbulence, mixing, potential and kinetic energy. The potential energy in the influent creates what is known as the density current. This current is caused by differences of density between the influent and the surrounding suspension. Horizontal density differences thus make the final clarifier a stratified flow phenomenon (Cordolia-Malina et al, 1979, De Vantier & Laroc, 1987, & Larsen, 1977).

Most of the sludge transport and most of the solids separation takes place in the density current. The heaviest part of the activated sludge settles out in the upstream portion of the settling tank, whilst the lighter portion is transported downstream. Sufficient time must be provided for the activated sludge flocs to traverse the density current and settle out before they reach the end of the tank. Density current height increases with increased hydraulic load, making the distance the sludge particles have to settle, before reaching the sludge blanket, longer. If sufficient time is not available before the density current reaches the

end wall (where it is reflected upwards and backwards towards the outlet weir) the effluent quality will deteriorate (Harremoes et al, 1993).

Turbulence and relatively high velocities can re-suspend flocs from the sludge blanket and transport them to the outlet weirs. This adds to the effect of the density current, and results in the lighter part of the activated sludge accumulating at the influent end. This often influences effluent quality and reduces plant efficiency due to sludge loss (*sludge blanket breakthrough*). The principle is similar for circular settlement tanks. Increased turbulence adversely affects the settling process. The areas worst affected within a circular settlement tank are towards the peripheral walls. The central area is protected by a diffusion gate.

The performance of this unit process is therefore susceptible to disturbance caused by combined (storm) loading. This is because the characteristics of the sludge can significantly change throughout a combined loading event from those which the tank was designed for.

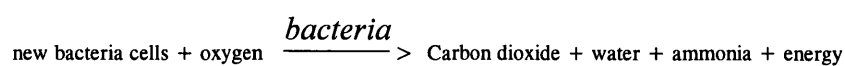
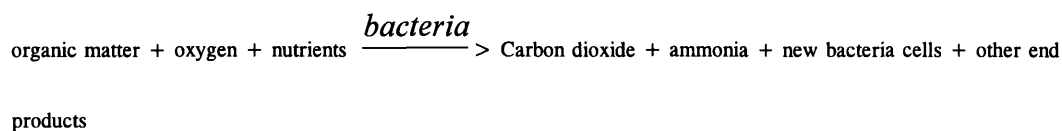
3.3 Biological Unit Processes

The objectives of the biological unit processes are to reduce the organic content (Carbonaceous BOD) and, in many cases, nutrients such as nitrogen and phosphorus. Nitrogen and Phosphorous removal have been given particular attention in recent years, primarily because of their effects in accelerating eutrophication of watercourses and promoting aquatic micro-organism growth. Nitrification of wastewater is required in many cases to reduce ammonia toxicity or to lessen the impact of oxygen depletion in flowing streams or estuaries (Metcalf and Eddy, 1991).

3.31 Activated Sludge

The Activated Sludge Process (ASP) is the most common treatment process for the purification of domestic waste and is referred to overleaf.

Biological degradation is second stage treatment which follows the primary treatment stage of sedimentation. In the ASP, heterotrophic bacteria are responsible for the degradation of the organic waste. Heterotrophic bacteria use complex carbon compounds as substrate (food). The bacteria culture carry out the conversion in accordance with the stoichiometry as shown below:-

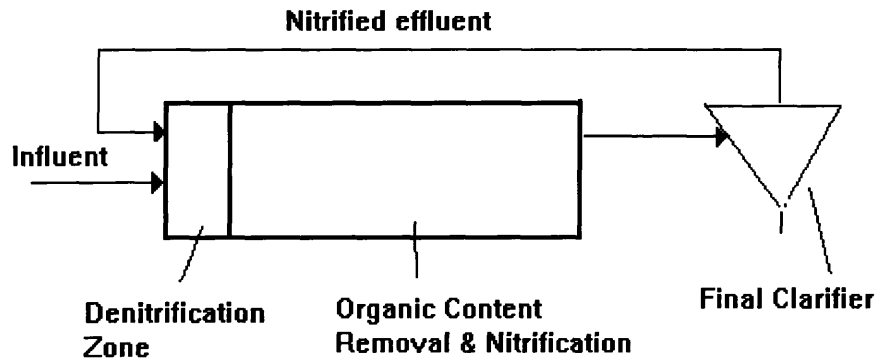


Autotrophic bacteria utilise carbon dioxide as a substrate source. It is these bacteria which are responsible for the nitrification phase of the nitrogen removal process. The nitrogen content of the wastewater is removed in a two part process known as nitrification/denitrification. Nitrification generally occurs after organic matter removal because autotrophic bacteria (which are responsible for nitrification), have lower growth rates than the heterotrophic bacteria (which degrade the organic waste). The two bacteria are therefore in competition for the available oxygen.

The nitrification process does not remove the nitrogen from the wastewater but does however reduce its oxygen demand. Nitrogen removal occurs during the denitrification phase. Heterotrophic bacteria (which require complex organic compounds for substrate) and anoxic (no molecular oxygen present) conditions are required for denitrification. Nitrified wastewaters are low in complex organic compounds, the available substrate is utilised in the degradation of the organic waste. A plant layout similar to the one shown below (fig. 3.2) is required for a denitrifying WTP.

Fig.3.2

Schematic Diagram of a Denitrifying WTP



Heterotrophic bacteria are also responsible for the removal of phosphorous. In order to achieve maximum phosphorous removal an oxic zone (oxygen present) requires to follow an anaerobic zone (no oxygen present). This allows the bacteria to uptake (oxic zone) and release phosphorous (anaerobic zone) above normal levels.

3.4 The Effects of Transient Loading on Unit Processes

During rainfall events, treatment plants receive influent volumes of much greater magnitude than those experienced under dry weather conditions. This is never more so than for a treatment plant which has been upsized/upgraded. The altered loading conditions resulting from combined inflow influence the efficiency of the various purification processes, thereby affecting the effluent quality of the treatment plant. In order to protect the receiving watercourse, a near constant level of purification is desirable for all stages of treatment. The only way to achieve this, however, is to maintain the theoretical steady state condition:

$$\text{i.e. } \frac{\partial C}{\partial t} = 0. \quad \text{eqn. 3.1}$$

where

C = pollutant concentration in reactor, and

t = time.

This condition is *assumed* to exist under dry weather conditions, but not under combined inflows/transient loading. The requirements are therefore in opposition to the actual conditions which occur (Durschlag et al, 1991), (Kollatsch, 1993).

3.41 Primary Clarifier Under Combined Loading

The critical problem in relation to combined waste water loading is the varying retention behaviour in respect of quantity and concentration. Dry weather concentrations remain for about the retention time of the tank but the flow rate at the outlet is increased within minutes. The dilution of dry weather concentrations is not realised until later (approximately the retention time of the tank). As a result, a high load of pollution is pushed out of the primary clarifier into the biological reactor.

The positive influences combined inflows exert on the primary clarifier are that the mineral content in the solids from the sewerage system is increased, thus aiding sludge thickening characteristics, and the different flow and concentration retention times in the primary clarifier help dampen the first flush effect from the sewerage system (Durschlag et al, 1992).

3.42 Secondary Clarifier Under Combined Loading

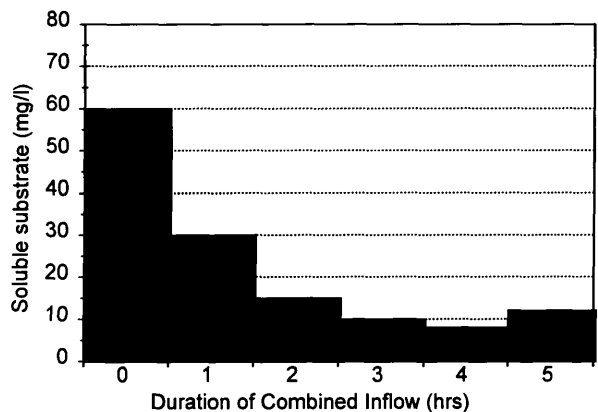
Wet weather loads to a wastewater treatment plant affect the secondary clarifier a short while after the rainfall event has begun. The effect on the effluent can be quite significant. Under wet weather loads the mass-transfer of the solids to the secondary settling tank can increase to a level typically beyond the capacity of the tank. The sludge blanket level increase generally occurs within approximately thirty to sixty minutes of the combined inflow (Lumley and Balmer, 1987). If the tank is too shallow then the increase of accumulated solids and the sludge blanket height will affect the effluent quality. It is therefore apparent that settling tank depth is of utmost importance for effluent quality under wet weather loading. In general, the problems associated with final clarifiers during combined inflows are concerned with the sludge blanket

height (at beginning of the event) and the variation in physical characteristics of the influent solids (settlement properties etc.).

3.43 Biological Reactor Under Combined Loading

The nitrifying bacteria are slow to respond to changes in influent characteristics and so load peaks resulting from flushing of the primary tanks can pass straight through into the effluent (Londong, 1994). Due to reduction in suitable substrate with storm duration (Fig 3.3), the denitrifying process would also be affected.

Fig. 3.3 Influent Soluble Substrate (Combined Loading)



Source:- (Durschlag et al, 1992)

The reduced substrate concentration over time, and the increased oxygen concentrations in the inflow (due to turbulence) exert a negative effect on biological phosphorous removal.

The efficiency of organic matter removal should not be seriously affected during combined loading, provided the aerators supply sufficient amounts of oxygen. The critical problem can occur after the rainfall event, when dry weather concentrations re-establish. This is because a significant portion of the active bacteria (known as the activated sludge), can be displaced into the secondary clarifier during periods of increased hydraulic loading (Durschlag et al, 1992), (Harremoes et al, 1993) and (Henze, 1987). When dry weather conditions re-occur, the reduced activated sludge mass within the reactor may

be insufficient to optimally treat dry weather flow concentrations (Henze, 1987) and (Durshlag et al, 1992). This effect is exacerbated if the final clarifier loses displaced activated sludge via the outlet weir. Regeneration of the active biomass, to an optimum level within the clarifier, may take a very significant time to occur, even months (Henze, 1987). The problem of sludge loss is documented as being more common in shallow settlement tanks (1.2 - 2.72m), as such tanks do not have the depth to accommodate the increases in sludge blanket levels which occur during storms (Parker, 1983). Sludge loss problems can also be compounded by variations in the sludges' settlement characteristics from those used in the initial design of the tank. This is a natural phenomenon and is a consequence of the variability of the concentration and specific gravity of the sediment passing through the primary tanks, the character and amount of industrial waste contained in the wastewater and the composition of the microbial life of the floc (Metcalf and Eddy, 1991).

3.5 Summary

Combined inflows can have a significant impact on the performance of a wastewater treatment plant. The individual problems are summarised below:-

- A high load of pollutants can be displaced from the primary sedimentation tank into the reactor, by the influent combined/transient wastewater.
- The increased hydraulic loading on the biological reactor can displace a portion of the activated sludge into the final clarifier. A certain time is required before the displaced biomass can be returned to the reactor. This could possibly result in poorly treated DWF.
- As the final clarifier is a unit process extremely susceptible to hydraulic disturbance, the displaced activated sludge can be lost over the outlet weirs. If this occurs the concentration of activated sludge remaining in the biological reactor may be insufficient to optimally treat the dry weather flow concentrations which rapidly re-establish after the rainfall event.

- Prolonged periods of diluted influent can adversely affect the activated sludge and reduce the efficiency of the WTP.
- Biomass requires a 'regeneration time'. The duration required for biomass regeneration may be very significant, depending on the severity of the rainfall event and the disruption caused at the WTP. Bacteria responsible for nitrification (autotrophic bacteria) have much slower growth rates than heterotrophic bacteria and therefore require longer regeneration times. Consequently, WTPs which are required to nitrify and/or denitrify may be significantly affected by activated sludge loss.

Chapter Four

The Receiving Watercourse

4.1 Introduction

As treatment plant performance can be upset by the introduction of in-sewer storage, an optimum balance should be found for reducing upstream discharges, and increasing discharges downstream. The assimilative capacity of each receiving water must be considered (i.e. it is not possible to presume that a reduction in total load must provide an improvement). For receiving waters which receive effluent from both the WTP and the CSO's then it is, of course, the total discharged load which must be considered, although this depends on where the emissions are in the watercourse. In essence each sewerage system, wastewater treatment plant, and their receiving waters, must all be viewed holistically, if the Best Practical and Environmental Option (BPEO) is to be found.

4.2 Integration

Traditionally sewerage systems and wastewater treatment plants have been planned, designed and operated by specialists, who work in relative isolation from each other (Capodaglio, 1994). This should not be the case (Harremoes, 1989). Sewerage systems present operational problems, and upgrading measures are generally based on obtaining a balance between flooding and overflow operation. The trend has been to carry out analysis with little or no regard for the treatment plant. Likewise, treatment plants have been designed without proper regard for the variability of combined inflows arriving at the works, with the process units sized using general guide-lines (e.g. treat an x multiple of dry weather flow). Both situations may lead to an overall

inefficiency of system performance and ultimately a waste of financial resources.

It is, in defence, fair to say that these practises were carried out, not in blatant disregard of receiving watercourses, but rather, as a direct consequence of a lack of engineering tools. This is because engineering practises, by definition, must be formulated, in terms of available 'know how'. Utilising only hand calculations and previous experience, no means were available to initiate sophisticated designs of CSOs (**Harremoes, 1989**). Consequently, the designs were based around simple criteria e.g. 1:6 dilution ratio of dry weather to storm flows. These criteria were therefore defined, for simplicity, rather than permissible pollution. The tools now exist to re-evaluate the design procedures, and thus base CSO spills on discharged load, and also to base the design of WTP process units on the required effluent, necessary for the receiving water quality, given the most probable quantity and quality of the hydraulic load emanating from the sewerage system (**Lijklema, Tyson and Lesouef, 1993**). There is now the capability to define complex rules, attained via complex analysis. In order to do this effectively however, it is important to understand the impact sewerage discharges (**Field & Turkeltaub, 1981**) and the WTP effluent have on receiving water quality. Considering CSO discharges, localised deposits of heavy organic and mineral particles can occur on the river bed around the systems overflows. These deposits can act as a further source of pollution if spread downstream by increased river flows (**Arsov, 1993**). It is understood that the capacity of a natural water to receive discharged loads is a function of morphology, flow rate, natural impurities and also intensity, duration, and frequency of rainfall, as these characteristics influence the discharged pollutant load. Essentially, however, the pollutant effects, be they from a CSO or a WTP, can be categorically divided into two simple groups; acute pollution and accumulative pollution.

4.3 Acute Pollution

Acute pollution is caused during and after a discharge but has no long term effects (**Harremoes et al, 1993**). Therefore after a rainfall event, acute

pollution will have only short term consequences. Examples of such consequences would be oxygen depletion from organic discharges, bacterial pollution from pathogenic bacteria and fish toxicity from ammonia discharges.

With regard to acute pollution, it is the extremity of an event which is important - not the accumulative load. Loads are frequently given as mass discharge per year for BOD or discharge requirements given as average concentration. Meaningful information on discharge from acute pollutants should, however, be given as extreme values, characterised by a statistical expression, like return period. This also holds true for discharge standards e.g. discharge permits should be 'maximum per event' or 'per day', with a return period of say five years {such an approach has been adopted within UPM - intermittent discharge standards state that the dissolved oxygen within a receiving watercourse should not fall below 'x' mg/l for duration 'y' more than 'z' times per year (FWR, 1994)}.

It is the mass discharge which is important. Effluent from treatment plants, discharge from *separate* sewer systems and all other sources of pollution e.g. agriculture, should all be considered in a similar manner (Harremoes et al, 1993).

4.4 Accumulative Pollution

The second category of pollution is characterised as having limited immediate effects, but which gradually build up over a period of time becoming detrimental to receiving water quality. These pollutants are accumulative pollutants.

Examples are:-

- Nutrients causing eutrophication of lakes
- Accumulative toxicants e.g. metals and specific organics.

It is the total discharge to the receiving water that is of importance here, and an extreme event is usually of little significance. Consequently, it is the yearly discharged load which is of consequence.

4.5 Conclusions

- If combined sewer overflows discharge to a different receiving watercourse than the WTP the solution must consider the sensitivity of both receiving watercourses i.e. not total emissions.
- Meaningful information on discharge of acute pollutants should be given as extreme values, characterised by a statistical expression, like return period.
- The long term performance of a drainage catchment as analysed by the German Total Emission Study group, does not therefore give all the required information with respect to acute pollution.

Chapter Five

Interaction Between Sewerage System & WTPs & Literature Review Summary

5.1 Introduction

The previous chapters discussed in detail the problems which combined flows exert in terms of the sewerage system, WTP performance and receiving water quality. This chapter draws upon the derived conclusions from each area to show that the interaction between the systems should, at least, be considered if full optimisation of the entire system is to be made. A description of the most prominent research carried out in this area has also been provided along with a summary of the research proposed to further the total emission knowledge base.

5.2 Sewerage Systems, Storage and WTP Performance

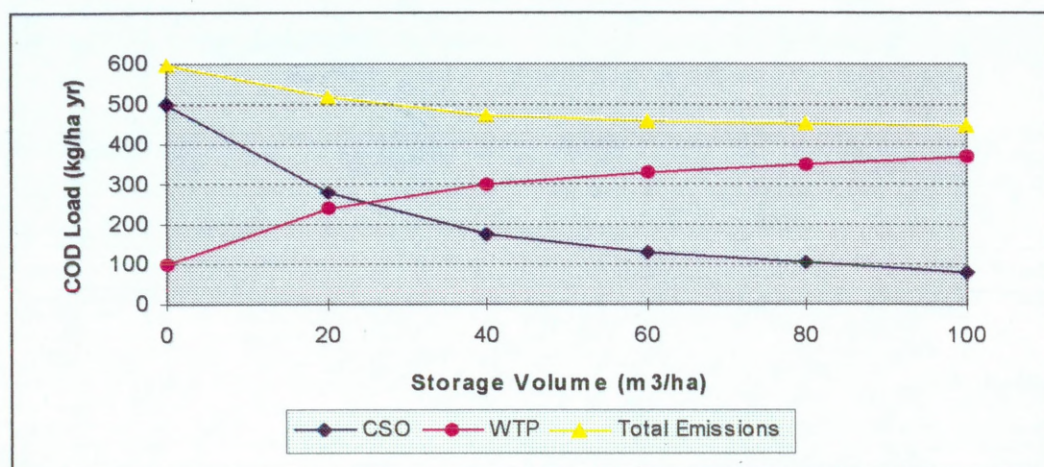
As storage basins prevent the hydraulic alleviation of a sewerage system by way of flooding or overflow operation, greater overall storm volumes are conveyed to the treatment plant (**Henze, 1987**). Consequently, large storage basins can be of little or no benefit with respect to total emissions due to two main factors. Firstly, the prolonged diluted inflows, as a consequence of the controlled emptying of stored wastewater, place the WTP under full hydraulic loading for greater periods of time, thus increasing the general disruption at the WTP (**Durschlag et al, 1992, Bertrand-Krajewski et al, 1994**). Secondly, by prolonging the duration of diluted inflows the regeneration of the bacteria is delayed, as the bacteria's growth/reproduction constituents are denied. Days of degraded treatment performance may therefore pass after an upset incident before the plant can restore itself to normal functioning (**Howard, 1993**). In

more extreme cases steady state conditions within a treatment plant may take months to occur (Henze, 1987) without operator intervention i.e. reseeding of plant. The addition of large storage volumes may therefore in certain circumstances bring about no overall improvement with respect to total emissions.

5.3 Total Emission Study

A German total emission study group (Durschlag et al, 1992) have shown that by increasing in-sewer storage volumes above $20\text{m}^3/\text{hectare}_{\text{imp}}$ no great improvement is gained with respect to total emissions when considered annually (Fig.5.1).

Fig.5.1 Total Emissions versus Storage Volume



Source:- (Durschlag et al, 1992)

The study however was focused on the long term performance of the drainage catchment, as can be seen from the discharged load stated in kg/ha yr. This is because in Germany, emission standards for organic material, at the time of writing, were based on loadings per year (ATV, 1992). Unfortunately organic loadings per year give no real indication of the increase/decrease in total emissions for discrete rainfall events, which are of importance with respect to acute pollution. In addition, as the German standards are based on load per year (ATV, 1992), the conceptual sewer system model KOSIM, was developed explicitly for the purpose of analysing the long term performance of the

sewerage system (**Bertrand-Krajewski et al, 1994**). Consequently the KOSIM model is more suited to long term analysis than discrete event analysis. This annual loading approach can however be considered to be realistic as current knowledge about the key in-sewer processes is less than desirable to achieve the deterministic models required for detailed event analysis (**ATV, 1992**). Unfortunately, and notwithstanding current limited knowledge, the problems of acute pollution still remain and cannot be ignored, especially if the Environmental Quality Objective (EQO)/Environmental Quality Standard (EQS) approach is to be maintained.

5.4 Dilution Ratio

It is also important to determine the optimum storm flow to dry weather flow dilution ratio to be treated at any WTP (**Lindholm, 1976**). This can be done using dynamic computer model simulation. In many countries, with the exception of the U.K. and France, the dilution ratio is set up to twice the average peak dry weather flow, whereas in France and Great Britain the treated flow is up to three times the average dry weather flow (**Harremoes et al, 1993**). Studies by **Otterpohl et al, 1994** have confirmed that both storage and varying dilution ratios treated have a significant effect on total load discharged from the system. If a system is to be optimised these factors must be investigated. The German approach to wastewater treatment is to biologically treat 2DWF. Consequently, the optimum storage figure of 20m³/ha is related to both annual loadings and WTP's which purify 2DWF. This figure is not directly relatable to U.K. or French WTP's.

5.5 WTP Control

Operational control of the WTP is also highly relevant. **Kappler and Gujer, (1993)** state that poor plant operation affects many existing WTPs. Oxygen electrodes are not maintained, insufficient control is given to solid retention time and sludge recirculations are not controlled optimally. Any detrimental effect of storage on WTP performance will be magnified by poor WTP operational control. The above authors therefore state that as a general strategy

the improvement of effluent quality from wastewater treatment plants should have priority over the reduction of pollutant loads from the combined sewage.

5.6 Summary

The effectiveness of a conventional sewage treatment plant is dependent upon the quality and the rate of raw sewage flow and the operational characteristics of the WTP. Sudden changes in flow rate, or in the quality of the raw sewage will degrade the level of treatment and so significant changes within the sewerage system will directly affect the degree of treatment received at the treatment plant.

Sewage treatment efficiency generally decreases rapidly with high wet weather flows and is slow to recover after the rain stops. Thus after a given rainfall event the accumulated pollution loading to the receiving watercourse from the sewerage system and treatment plant, may be greater than the total load discharged from these systems before in-sewer storage was utilised. It is therefore stressed that during periods of excess runoff, combined sewer overflows directly add to the pollution of the receiving waters but also indirectly reduce pollution by protecting the efficiency of the treatment plant. It may therefore be preferable to deliberately spill to protect the plant. The question is how much, where and when (real time control)? This question has not, as yet, been answered in a satisfactory manner for general application.

5.7 Sewerage System and WTP Interaction Conclusions

- Sewerage systems and treatment plants have, in the past, been considered in relative isolation of each other. This should no longer be the case. In order to protect receiving watercourses, criteria must be defined, which will allow sewerage managers to improve the performance of their *entire* systems, albeit, without entailing excessive costs.
- The success of an improvement/rehabilitation measure can only be assessed by analysing the total loads emitted from the sewerage and WTP. The

consideration of individual aspects e.g. the storage of combined wastewater with the aim of minimising direct emissions from the storm water overflows, may produce only an incomplete picture of the total load imposed on a receiving water during and after rainfall.

- In-sewer storage volumes can have a significant benefit with respect to overflow volumes. They can however adversely affect wastewater treatment plant performance.
- Insufficient work has been carried out quantifying best in-sewer storage volumes with respect to WTP performance and acute pollution.
- Typically treatment plants purify a certain portion of the combined flow, generally chosen as two or three times the dry weather flow. This may not be the best ratio for specific plants and may, as a consequence, result in operational problems. Evidently this should be investigated in order to provide optimum protection of receiving watercourses.
- Improvements in operational procedures at poorly controlled WTPs should have priority over the reduction of pollutant loads from the sewer. This is because the detrimental effect of storage (on WTP performance) will be magnified by poor WTP operational control.

5.8 Overall Literature Review Summary

The integration of planning and management of wastewater systems has been poor almost everywhere (Lijklema, 1992). The reason for this has primarily been because those responsible for the sewerage system, waste water treatment plant, and the receiving waters have acted in relative isolation from each other. This has been the case not only in the sense of organisation (although less-so in the UK than throughout the rest of Europe) but also in that the understanding of the physical interactions of a catchment have been less than desirable. A

strong relationship is known to exist between effluent quality and quantity and the components within a drainage catchment, and integrated management and planning may be required if more sustainable solutions to drainage catchment problems are to be found.

5.9 Previous Research and Shortfalls

The most extensive work in this area **Durschlag et al, 1991** concluded that storage volumes above $20\text{m}^3/\text{ha}_{\text{imp}}$ (Fig. 5.1) scarcely reduce the total emissions discharged from the catchment (measured by COD/ha yr). Although this research focused attention on the potential necessity of total emission analysis for cost effective solutions, the general application of the work, was nevertheless restricted. There are three main reasons for this;

1. The study was carried out using only one drainage catchment and thus the site specifics of the interaction problem was not taken into consideration; each sewerage system and WTP are likely to interact in a unique way, consequently, a large storage volume in one particular catchment may well have profound effects upon the performance of the WTP, whereas the same storage volume in other catchments may not exert such pronounced effects. The reason being that the interaction is dependent upon a number of site specific factors such as climatic conditions (rainfall profiles), type of effluent being conveyed to the plant (e.g. industrial - which can affect the bacteria's activity) the loading state of the plant (overloaded etc.), general WTP efficiency and also operational procedures adopted at the plant. As these problems are related to site specific circumstances they limit the applicability of globally defined rules.
2. In Germany up to twice peak DWF is treated physically and biologically whereas in the UK six average dry weather flow can be treated physically and up to three times the average DWF is treated biologically (this approach is also adopted in France). Due to the differences in operational procedures between U.K. and German plants it is likely that different total emissions

would be discharged from the respective plants for the same given influent conditions. Consequently, it is also very likely that the range of suitable storage volumes ($0\text{-}20\text{m}^3/\text{ha}_{\text{imp}}$) is applicable only to WTPs which utilise the same operational control strategies as the Germans.

3. Although the research highlighted that differences in total emissions would result via the utilisation of varying storage volumes, no suitable information was provided with respect to the benefit these storage volumes would have in terms of acute pollution. This was because the analysis was carried out using a yearly analysis period. As COD is a measure of organic matter, which gives rise to receiving water quality problems during and immediately after a rainfall event, a more appropriate time-scale for the measure of organic loading would have been much shorter. The reason for this is that the total number of wet weather hours in a year are typically very small in comparison to the total number of dry weather hours (in notoriously wet countries such as Denmark, rainfall is known to occur for only 5% of the time, (Henze, 1987). Consequently, peak loads in organic matter (from the sewerage system and treatment plant), which could produce significantly detrimental acute pollution effects within a receiving watercourse, would not be readily observed using a yearly analysis period. It was therefore concluded that the German research work was also, in its current form, of limited use with respect to acute pollution analysis.

In order for engineers to know which ranges of storage volumes would provide solutions to receiving water quality problems, in terms of acute pollution (and to know which ranges of storage would exacerbate the problems, or provide no significant benefit) more detailed analysis with respect to the boundary conditions of the storage/total emission problem therefore required to be carried out. This analysis was to be done following the aims and objectives as outlined in chapter one. New research in this area would also promote the production of cost effective solutions, which due to limit financial resources, are now essential. A detailed description of the proposed method is presented in chapter sixteen,

however, before this method was fully developed, subsidiary aims, as highlighted in chapter one, were to carry out an evaluation of the available sewer flow, sewer flow quality and wastewater treatment plant models. It was deemed pragmatic to carry out the modelling evaluation prior to the development of the method to ensure the extent of the limitations and/or the capabilities of the models were fully understood. This would prevent an overly simplistic, or an overly complex method being developed as a consequence of under or overestimating the capabilities of the available packages.

The models which were evaluated are listed below:-

Sewer Hydraulics:-

Hydroworks-PM

Sewer Flow Quality:-

MOSQUITO, Hydroworks-QM

Waste Water Treatment Plant Performance:-

STOAT, GPS-X

These models were chosen primarily for reasons of availability and because they were, at the time of construction, recognised as the state of the art ‘integrated management’ modelling tools. The catchment which was chosen for the analysis was the City of Perth, Scotland. This location (which is described in the following chapter) was chosen for the simple reason that a previous research programme (**Petrie, 1997**), had resulted in the production of a verified hydraulic (WALLRUS) model, a calibrated sewer flow quality model (MOSQUITO - DWF only), and a partially calibrated WTP model (STOAT). The adoption of these packages in this project significantly reduced the project time-scale. No explicit consideration was given to the receiving watercourse as the large assimilative capacity of the River Tay meant that the development of a receiving water quality model could not be justified. Consequently, the integrated analysis was concerned principally, with a technical appraisal of sewerage system and wastewater treatment plant performance.

Analysis was also carried out to determine whether a more suitable multiple of DWF could be biologically treated at a WTP. This analysis was undertaken mainly for interest and is detailed in Appendix F.

Chapter Six

Catchment Description

6.1 Perth Sewerage System

The drainage area of Perth approximates to around 15km² and serves an estimated population of 45,000. The drainage system can be considered to be divided into the eight sub-catchments listed below

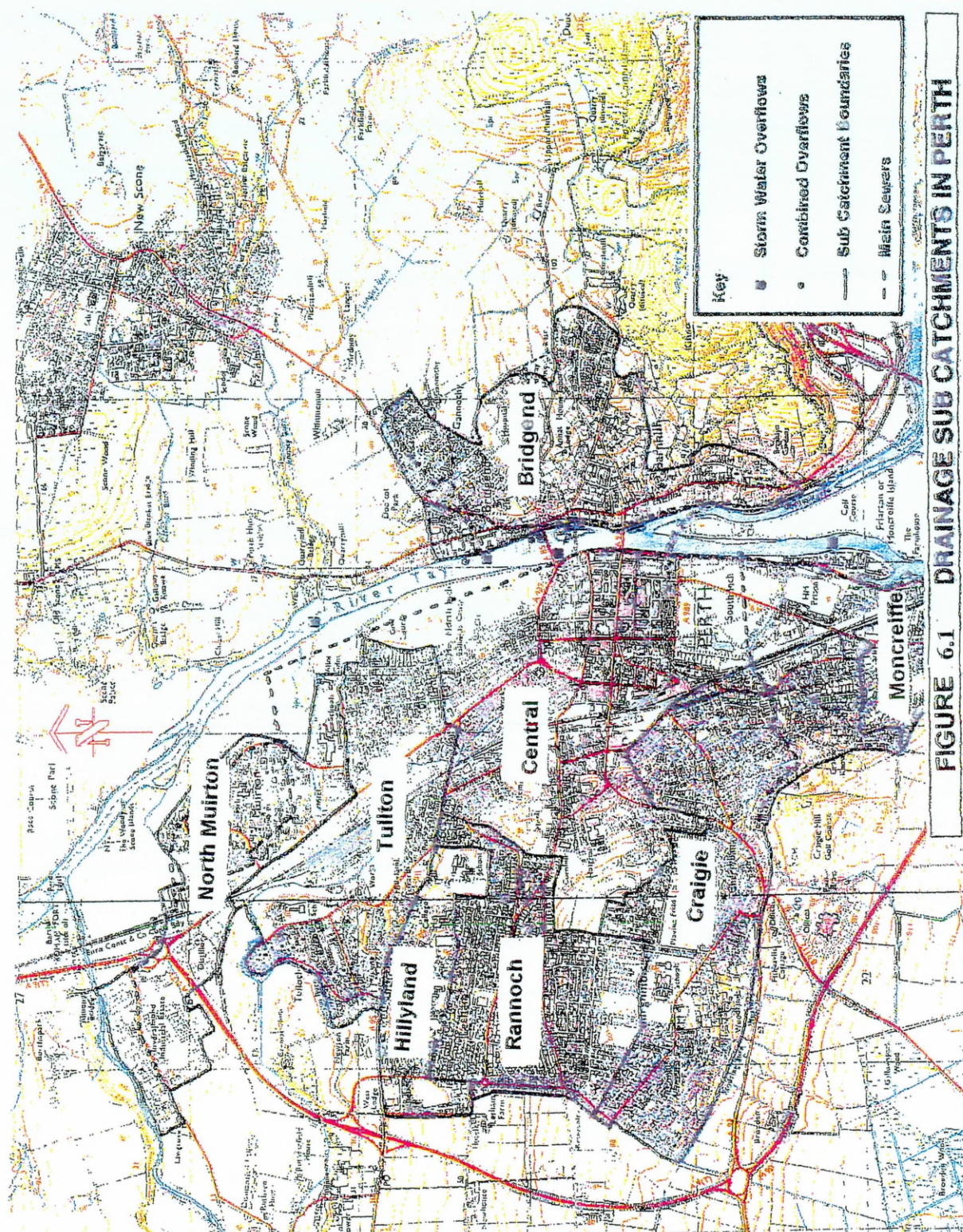
- Moncrieffe
- Craigie
- Rannoch
- Hillyland
- Tullton
- North Muirton/Inveralmond Industrial Estate
- Bridgend
- Centre

The sub-catchments are shown diagrammatically in Fig. 6.1.

6.2 North Muirton

The North Muirton sub-catchment is located to the North West of the City Centre and holds a population of approximately 4500. The sub-catchment is separately sewered with the surface water discharging to the Tay via two flap valved pipes. The foul sewers discharge to a 750mm diameter sewer which runs the length of the North Inch. This pipe connects into the main interceptor sewer after Perth Bridge. North Muirton contains a small industrial area. A larger industrial area is

Fig. 6.1 Drainage Sub-Catchments in Perth



located to the west in Inveralmond. This industrial estate contains various units from FMC (slaughter house) to Pullars (dry cleaners). The surface water from Inveralmond discharges to the River Almond.

6.21 Hillyland

The Hillyland sub-catchment holds a population of around 800. It is located between the Rannoch and Craigie sub-catchments and employs separate and partially separate sewerage systems. The sub-catchment is mainly residential, however small industrial units are located to the west. Old housing exists along the Crieff Road. Newer developments can be found on the steep hillside to the South of the catchment. The newer housing is serviced by a separate system with the storm water runoff being discharged into the Town's Lade.

6.22 Craigie

The Craigie sub-catchment is located to the south west of the city centre. The sub-catchment holds a population of around 7500 and is separate, partially separate and combined. The sewerage in the west is principally separate, however this progressively changes to combined moving toward the east. The storm water is discharged into the Scouring/Craigie Burn via six 'hole in the wall' overflows (Plate 1 - Appendix A). These overflows are located along the length of Windsor Terrace. Operation of these overflows has not been witnessed, however, aesthetic pollution can be encountered around these overflows, indicating their operation.

6.23 Rannoch

The Rannoch sub-catchment, which is located between the sub catchments of Hillyland and Craigie, is partially separate and predominately residential. No industry exists. The population of this subcatchment is approximately 6500. The surface water system gravitates to a culverted watercourse known as the Goodly Burn. This burn runs underneath the Crieff Road and discharges to the Town's Lade at the rear of the Fairfield estate.

6.24 Bridgend

This sub-catchment is located to the east of the River Tay and is made up of two residential areas. The population is around 2600. The sewerage system is combined and gravitates to the Willowgate pumping station, where the wastewater is pumped through a rising main attached to the railway bridge. The flows discharge into the interceptor sewer in Tay Street.

Four CSO are present in this sub-catchment. The first is located in Mansfield Place. This overflow operates frequently.

The next overflows is a high level relief pipe which discharges to the River Tay at the rear of the bakery in Main Street (Plate 2 - Appendix A).

The third CSO is located opposite Stanners Island in the area adjacent to the small slipway. This CSO is a high sided weir.

The last overflow in the Bridgend sub-catchment is located prior to the Willowgate pumping station (Plate 3 - Appendix A). This CSO is a single sided weir with rotating disc screens. The point of discharge is approximately 500m downstream of the pumping station.

6.25 City Centre

The city centre sub-catchment is defined by the boundaries of its neighbouring sub-catchments. The population is around 6000. The sewerage is combined and due to the slack gradients within this sub-catchment many of the pipes suffer from sedimentation. The area is residential in the outskirts and commercial in the centre.

6.26 Moncrieffe

Moncrieffe is located to the south of the city centre. It is a predominately residential sub-catchment with a combined sewerage system. The sub-catchment outfalls just downstream of the South Inch pumping station.

There are two culverted watercourses present in this sub-catchment. These streams meet on the west side of Edinburgh Road where they combined into one pipe. This pipe crosses the Tesco car park and the railway yard and finally discharges to the interceptor sewer at Friarton buildings.

6.3 General System Characteristics

Principally the sewerage system employs gravity to convey the flows to the main trunk sewer. Pumping stations at South Inch and Friarton utilise archimedian screws to help convey the flows to the wastewater treatment works at Sleepless Inch. The WTP consists of storm tanks, primary settlement, activated sludge aeration (plug flow) and final settlement tanks.

The Bridgend sub-catchment conveys flows to a pumping station, comprising submersible pumps, at Willowgate. The Willowgate pumping station then transfers the flows over to the main sewer via a rising main. This pumping station conveys a maximum flow of 100l/s with any excess sewage being discharged into the River Tay via a CSO at Willowgate.

6.4 System Deficiencies

There is evidence throughout the various subcatchments that the sewerage system in Perth is undersized and in need of upgrading (Plate 4 - Appendix A). The main causes are surcharging, due to the performance of the South Inch Pumping Station, and hydraulic inadequacies within the network (**Babbie, Shaw and Morton, 1993**). Developments over a number of years have also put such a strain on the existing network that the sewerage is simply unable to take the extra loads imposed on them by storms. Shallow gradients in certain areas also add to the problem. The following is a summary of the major deficiencies:-

6.41 Bridgend

Adjacent to the River Tay the length of sewer is of slack gradient and employs a number of overflows to discharge excess flows. Not all of these overflows utilise flap valves and are positioned at such a height that allows the Tay, at rare flood levels, to 'back up' and surcharge the system.

Due to the slack gradients surcharging can also occur at periods of low river flow. This constitutes a particular problem as manhole covers can be forced off by high pressures, which as a consequence, can result in raw sewage on the

surrounding land. Overflows in this sub catchment are also prone to discharge to side branches of the Tay which may have low volumetric flow rates during summer conditions, causing visual and possibly biochemical pollution (Plates 5 and 6 - Appendix A) .

The average dry weather flow at this subcatchment's outfall was logged at approximately 17l/s. The CSO at Willowgate operates when flows are in excess of six dry weather flow, with the surplus being transferred to an outfall some distance away from the pumping station. This overflow has recently been reconstructed.

6.42 North Muirton

The North Muirton Housing Estate is separately sewered with the storm water being discharged into the River Tay via twin outfall pipes. Flap valves are used at the outfalls to prevent the Tay backing up the outfall pipes. These flap valves have, in the past, been held open by accumulated debris. In January, 1993, surface flooding was observed in North Muirton prior to the river levels overtopping the North Muirton embankment. Reports state that flows were being discharged from the drainage system via manholes in the vicinity of Bute Drive (**Babtie, Shaw and Morton, 1993**). However, due to the nature of ground levels within North Muirton, this problem may occur solely as a consequence of high river levels (e.g. tidal effects etc.).

6.43 Perth Centre

Surcharging of the main sewer along North Inch, South Inch and Tay Street is known to occur in times of heavy rainfall with water levels as far as the school at North Muirton being affected. Babtie Shaw and Morton have documented, following discussions with NoSWA, that the probable cause may be due to the non operation of the overflow screen at the South Inch pumping station and the limited capacity of the trunk main.

As the screens in the pumping station are not cleaned they exacerbate surcharging within the system. During flood events the conditions in Perth centre

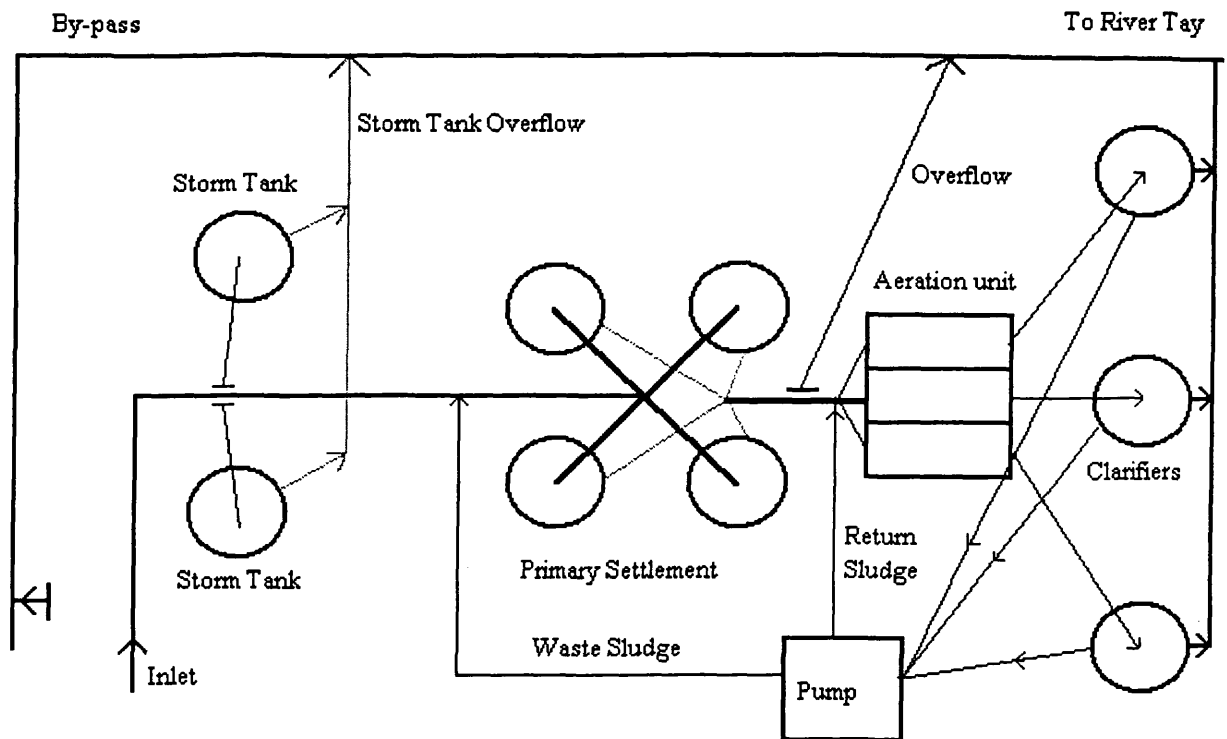
are exacerbated by this problem. The two main overflows at the South Inch (Plate 7 - Appendix A) and Friarton pumping stations are unable to discharge unless the water levels in the sewerage system are higher than the water levels in the Tay. This is a result of the flap valves being held closed.

A full hydraulic analysis of the Perth sewerage system was carried out as a part of the EPSRC CASE requirement. The results were detailed by **Jack, (1995)**.

6.5 Wastewater Treatment Plant

The WTP serving the City of Perth is located to the south east of Perth in an area known as Sleepless Inch. The Sleepless Inch WTP is designed to provide primary treatment for up to 6 times dry weather flow and secondary treatment, via a conventional activated sludge unit, for up to three times dry weather flow. Storm tanks with half the capacity of the primary tanks provide extra storage in wet weather. The works accepts tankered-in sludge from the surrounding area. These are added upstream of the works at Friarton pumping station (Plate 8 - Appendix A). Peak dry weather flow into the works is approximately 250l/s. After pre-treatment (Plate 9 - Appendix A) the influent enters 4 circular primary tanks each with a capacity of 1796m³ (Plate 10 - Appendix A). The 2 storm tanks are of the same design. Storm tank contents are returned to the head of the works. After primary treatment the settled sewage enters a single channel with a by-pass overflow. It then splits into 3 lanes (Plate 11 - Appendix A) and mixes with the return activated sludge (RAS) before entering the activated sludge plant (Plate 12 - Appendix A). Each lane is subdivided into three pockets, each with a capacity of 400m³, to give a total volume 3600m³. The operation of the ASP is controlled manually through regulation of the return and waste activated sludge. There are three secondary clarifiers (Plate 13 - Appendix A) each with a capacity of 1984m³. The mixed liquor is settled in the clarifiers and the supernatant mixes with any by-pass or overflow flows before draining to the River Tay. A diagrammatic layout of the works is shown in figure 6.2:-

Fig. 6.2 - Schematic Diagram of the Perth WTP



Chapter Seven

Sewer Flow and Quality Modelling Overview (WALLRUS & MOSQUITO - Theory)

7.1 Introduction

The model initially used for sewer flow quality in the Perth study was MOSQUITO. MOSQUITO is a deterministic model which operates parasitically from the WALLRUS hydraulics package. As previously discussed, the research project carried out by **Petrie, (1997)**, resulted in the construction of both these models (although the MOSQUITO model was handed over in an uncompleted state). This chapter briefly details the modelling principles of the respective packages as they were trailed for use within the project. Calibration of MOSQUITO under dry weather conditions is also discussed.

7.2 WALLRUS

WALLRUS consists of two sub-models; a surface runoff model and an in-sewer flow routing model. In the surface runoff model, net rainfall depth is calculated from a percentage runoff equation which is derived through regression analysis. The corresponding runoff is derived via the routing of this rainfall through a non linear reservoir. This process produces hydrographs which are then multiplied by the relevant contributing areas to form the surface runoff. In the routing model, the surface runoff hydrographs entering each pipe length are routed through the drainage network using a version of the Muskingum flow routing method. These equations are shown below:-

$$O_{t+\Delta t} = C_0 I_{ft+\Delta t} + C_1 I_{ft} + C_2 O_t \quad \text{eqn. 7.1}$$

where

$$C_0 = -[(KX - 0.5\Delta t) / (K - KX + 0.5\Delta t)] \quad \text{eqn. 7.2}$$

$$C_1 = [(KX + 0.5\Delta t) / (K - KX + 0.5\Delta t)] \quad \text{eqn. 7.3}$$

$$C_2 = [(K - KX - 0.5\Delta t) / (K - KX + 0.5\Delta t)] \quad \text{eqn. 7.4}$$

A comprehensive evaluation of the Perth WALLRUS model was carried out by **Petrie (1994)**. It was concluded that an adequate representation of the Perth drainage system could be obtained using WALLRUS, however mathematical instabilities could be expected in shallow gradient areas of the system. These instabilities were a consequence of the simple flow routing equations which were utilised by the package.

7.3 MOSQUITO (Theory)

MOSQUITO models the commonly used sewage quality parameters such as BOD, COD, TSS and Ammonia. The pollutants are modelled as being either attached to the sediment particles or being in solution. The quantity of pollutants attached to the sediment is defined via a fixed ratio referred to as the 'potency factor' (the potency factor is multiplied by the TSS concentration to provide a value for the BOD or COD content of the sediment). It is assumed that the pollutants attached to the sediments are eroded and deposited at the same rate as the sediments they are attached to and that the dissolved pollutants travel at the same velocity as the flow.

7.31 Sediment Transport Modelling

The Ackers/White formula, shown below, was used by MOSQUITO for the modelling of sediment transport. The model was first developed for sediment transport in open alluvial channels, however empirical adaptation for circular pipes has been incorporated. There are three main equations in the Ackers White approach:-

$$F_{gr} = u^*{}^n / \sqrt{gd_{35}(s-1)} [U / \sqrt{32 \log (12R_h / d_{35})}]^{1-n} \quad \text{eqn.7.5}$$

$$G_{gr} = C_{aw} [\{F_{gr} / A_{aw}\} - 1]^m \quad \text{eqn.7.6}$$

$$q_t = G_{gr} s d_{35}^{1/R_h} [U/u^*]^n [\{W_e R_h\}/S]^{1-n} \quad \text{eqn.7.7}$$

where

F_{gr}	dimensionless mobility particle number
G_{gr}	dimensionless solid flow number
g	acceleration due to gravity (m/s^2)
q_t	total solid flow (kg particles/kg water)
u^*	friction velocity (m/s)

U	mean flow velocity (m/s)
R_h	Hydraulic radius (m)
s	specific gravity of particles
d_{35}	particle diameter (35% in mass of particles have a diameter less than d_{35})
A_{aw}, C_{aw}, n, m	coefficients depending on dimensionless particle diameter
We	effective deposited sediment width (m) { $We=10 \times d_{35}$ }
S	flow section (m^2)

The equations are believed to produce reasonable estimates of suspended material movement, although, certain limitations have been identified (MOSQUITO User Manual, 1993):-

- Sediments in sewers are widely graded whereas the formula is relative to narrowly graded sediments.
- The formula was developed for steady state conditions whereas the conditions during rainfall events are unsteady.
- The formula was developed for alluvial rivers with an unlimited supply of materials available for erosion. In sewers there is only a limited supply of erodible material from the sewer bed.
- The formula ignores fine suspended load (i.e. the sediments in the silt/clay range).

7.32 Pollutant Origins and Modelling

The MOSQUITO model accounts for pollutants originating from surface runoff, inflow hydrographs and foul water being mixed and transported through the flow. The package however, does not include degradation of pollutants throughout their journey through the sewerage system, or any biochemical interactions between different pollutants. The assumption is made that these effects are insignificant due to the relatively short hydraulic detention times within the sewers.

7.33 Sediment Layers

Within MOSQUITO, sediments are considered to exist in two discrete layers; an active layer on the pipe invert in which fine sediment is stored in an unconsolidated state, readily available for transportation by the flow, and a consolidated storage layer in which a cohesive shear strength must be overcome

before it can be disturbed. The processes of sediment erosion and deposition occur between the flow and the active layer. The active layer is composed of particles of pollutants which have settled out of the flow during the dry weather (antecedent) period. A steady-state calculation is carried out to ascertain the amount of fine sediment which will build up in certain pipes within the sewerage network. This then may facilitate a first foul flush occurrence within the sewerage if the shear stresses are subsequently sufficient to cause entrainment. The consolidated storage layer represents sediment which has become consolidated over a period of time. It is composed of coarse and fine sediment fractions and contains polluted interstitial liquid. If the active layer is completely eroded away at any 'computational point' and the boundary shear stress of the flow is greater than the shear strength of the storage layer, then a small amount of the storage layer (10 mm) is assumed to be disturbed. If this layer is eroded and transport capacity still exists, another 10mm layer will be made available for erosion, and so on.

7.4 Perth MOSQUITO Model Construction -Dry Weather Flows and Quality

MOSQUITO contains default characteristics which have been obtained from various data collection exercises. However, as it is recognised that substantial variation exists in sewer flow quality from location to location, it was thought pragmatic to construct the Perth MOSQUITO model entirely using collected data.

7.41 Data Collection

The dry weather flow and quality input data were collected at the top of each sub-catchment at locations where no significant sediment build up was evident. Detectronic flow survey loggers and Epic portable wastewater samplers were used, the former logging flow depth and velocity at two minute intervals, the latter collecting twenty four samples, each hourly sample made up of four fifteen minute composites. The data collection exercise was carried out by the North of Scotland Water Authority (NoSWA), in conjunction with the University of Abertay Dundee (UAD).

7.42 Land Use Identification

An important stage in the construction of a sewer flow quality model is the designation of land uses. The purpose of identifying 'land-uses' is to associate different areas (e.g. residential, industrial etc.) with specific flow and quality characteristics. For the Perth catchment each separate sub-catchment was

allocated an individual land use as shown in table 7.1. The data below were obtained from the data collection exercise.

Table 7.1 Land Use Identification and Characteristics

Area	Land Use Number	Area (km ²)	Ave. Flow at Sub-Catchment Outfall (m ³ /s)	Flow/Area (m ³ /s/km ²)
Muirton/Inveralmond	1	0.693	0.015	0.022
Tullton	2	0.626	0.002	0.031
Rannoch	3	1.270	0.015	0.012
Bridegend	4	1.704	0.005	0.028
Craigie	5	0.169	0.012	0.079
Moncrieffe	6	0.139	0.002	0.017
City Centre	7	0.791	0.017	0.021
Carrier Pipes	8	0.000	0.000	0.000
Hillyland	9	0.483	0.054	0.011

The corresponding pollutant characteristics for each sub-catchment are shown in table 7.2. The data were obtained from the Perth data collection exercise.

Table 7.2 Dry Weather Flow Pollutant Characteristics

	Muirt'n	Tull	Rann	Bridge	Craigie	Monc	Cntr	Hillyland
Ave. TSS (mg/l)	456.4	229	299	315	315	168	456	230
Ave. COD Diss.	511	256	410	527	432	220	511	256
COD. Pot.Factor	1.944	3.15	1.04	1.5	1.37	1.208	1.94	3.15
Ave. Ammn	59.3	33	45.7	37	37	33.8	59.3	33
Time	Diurnal Multipliers (Hourly Concentration = Ave. Concentration x Diurnal Multiplier)							
09.00	1.090	1.582	1.182	1.528	1.312	1.304	2.04	1.582
10.00	1.034	0.874	0.924	0.481	1.230	1.219	1.54	0.874
11.00	1.437	0.746	0.952	0.885	1.023	1.159	1.2	0.746
12.00	1.532	0.756	0.786	0.722	0.923	1.123	1.07	0.756
13.00	1.588	0.831	0.940	0.808	0.903	0.777	0.94	0.831
14.00	1.118	0.700	0.718	1.575	0.985	0.720	0.86	0.7
15.00	0.890	0.939	0.713	1.789	0.812	0.994	0.90	0.94
16.00	0.932	0.8	0.799	1.99	0.878	0.944	0.94	0.8
17.00	0.985	1.146	0.685	1.15	0.843	1.520	1	1.146
18.00	1.133	0.912	1.112	1.59	0.928	1.164	1.07	0.91
19.00	0.979	0.818	1.219	1.2	1.025	1.174	1.09	0.82
20.00	0.963	0.707	1.04	0.85	1.036	1.027	1.06	0.707
21.00	0.847	0.846	0.884	1.05	0.94	0.915	1.08	0.846
22.00	0.786	0.743	1.041	1.11	0.978	0.836	1.11	0.743
23.00	0.761	0.795	0.932	1.03	0.954	0.654	1.22	0.795
00.00	0.649	0.773	0.933	1.366	0.933	0.961	1.17	0.723
01.00	0.572	0.629	1.092	0.000	1.063	0.716	1	0.627
02.00	0.503	0.628	1.318	0.000	1.729	0.329	0.61	0.267
03.00	0.472	0.000	0.000	0.000	1.779	0.232	0.41	0
04.00	0.450	0.228	0.000	0.123	1.434	0.163	0.22	0.228
05.00	0.473	0.201	0.000	0.847	0	0.171	0.26	0.2
06.00	0.413	0.322	1.730	1.292	1.717	0.672	0.43	0.322
07.00	0.809	1.392	1.164	1.726	0.6	1.146	0.76	1.392
08.00	1.169	1.704	1.152	2.090	0.939	1.282	1.81	1.71

Note:- data shown in table 7.2 are adjusted to account for the dilution affects of infiltration.

7.5 Dry Weather Flow Verification

Using the flows per area for the various subcatchments as shown in table 7.1, MOSQUITO was able to simulate the hydraulics of the Perth system (Appendix D figure D.1). The only problems encountered were caused by instabilities within the WALLRUS hydraulic package. To remedy this problem the "backwater flags" required to be removed.

The modelled flows at the outfall (Friarton pumping station) followed the logged values closely. The maximum modelling error was however +60%. This occurred in the early hours of the morning (5am). Total volume was over-predicted by 6.1%. With respect to the peak, a modelled peak of 240l/s was obtained at 11am, whereas the observed peak was 225l/s, at 1pm. This provided an error of +6.6% with a time lag of 2 hours.

7.51 DWQ Verification - TSS

Following the recommended procedure (**MOSQUITO User Manual, 1993**) for Dry Weather Quality (DWQ) verification, the sediment transport model was given first consideration. From observation of initial calibration attempts it was apparent that too much sediment was settled out of the flows. Consequently, the density of the sediment was decreased from the default value of 1200kg/m³. Various trial values were used however the most appropriate results were obtained when the density was reduced to 1010kg/m³ (figure D.2). The maximum modelling error was -92% at the system outfall. This error occurred at 6am. The total TSS load was however under-predicted by only 9%. The peak sampled TSS concentration occurred at 10am and was 515mg/l. No time lag was believed to exist and therefore the modelled prediction at this time step was 230mg/l. This provided an error of -55%.

7.52 DWQ Verification - COD

The next stage in the verification process was the consideration of COD. The initial runs showed very poor correlation between measured and predicted data sets. It was noted that the modelled dissolved fraction of COD showed good correlation with the observed total COD, yet when MOSQUITO added on the attached pollutant concentration the final correlation became very poor. From this it was deduced that MOSQUITO was using the input COD value as a measure of the dissolved COD component, then adding on the sediment attached component to provide the total. This was contradictory to the information provided within the **MOSQUITO User Manual (1993)**, however the hypothesis was substantiated by WRc who also discovered the problem (**Gent, 1994**). The input files were therefore revised and more suitable fits were obtained at each of the sampling points and the system's outfall (figure D.3). The modelled error at the system outfall with respect to the total COD load was +27%. A maximum error of +94% occurred at 10am. The observed peak occurred at 3pm and was 718mg/l. The modelled peak occurred one hour later and was 711mg/l. This produced an error of -1%.

7.53 DWQ Verification - Ammonia

Verifying for ammonia did not constitute a problem as ammonia is dissolved and therefore essentially dependent upon the accuracy of the flow fit (figure D.4). The modelled error at the system outfall with respect to the total ammonia load was +12%. The maximum modelling error occurred at 4pm and was +61%. An observed peak concentration of 36mg/l was obtained at 9am. An arguable time lag of 1 hour was present, which produced a modelled peak of 32mg/l at 10am. Based on these figures the error with respect to the peak was calculated as -11%.

7.6 Summary

As no great problems were encountered during the DWF verification process it was concluded that the ability of MOSQUITO to model the advection and dispersion of pollutants throughout the Perth sewerage system was reasonable for dry weather flows. However, it was believed highly probable that significant problems could have been encountered if the default pollutant characteristics had been used in preference to site specific data. This is discussed in greater detail in section 7.9

7.7 Default Values and Site Specific Data

It was observed from the data collection exercise that the quality of sewage throughout the Perth system varied from location to location. Consequently, the utilisation of the default values proposed within the model would have increased the potential for erroneous assumptions being made during the calibration stage.

The concept of utilising average default values is also a practice which requires to be questioned. This is because the variability of sewer flow quality from location to location suggests that true average characteristics would not actually exist. Consequently, developing a large data base of default values, averaged from many data collection exercises would only serve to vary the average characteristics each time a new data set is added. If this is taken into consideration it becomes evident that site specific data are required and not averages, as such values will necessitate adjustment by the user to improve model calibrations. In some cases unmeasurable parameter adjustments are justified e.g. shear strength and settling velocity (Crabtree et al, 1993). However, the adjustment of average pollutant values to obtain improved fits is in essence a force fitting of the model, unless the adjustments can be validated by comparison with site specific data.

Table 7.3 represents a comparison of typical sewer flow pollutant data collected in Perth with both the original and revised default values (the original default values proposed within the MOSQUITO were revised as the data-base grew (**Gent et al, 1994**), thus demonstrating the point discussed above). Sub-catchments Moncrieffe and Rannoch were chosen arbitrarily and are characteristic data.

Table 7.3 Comparison of Site Specific Data and Default Values

	Moncrieffe observed data	Rannoch observed data	Original default data - MOSQUITO	Revised default data-MOSQUITO
TSS (mg/l)	168	299	240	250
BOD Dissolved (mg/l)	57.2	91	160	210
BOD Potency Factor	0.625	0.458	0.56	0.03
COD Dissolved (mg/l)	220	410	190	455
COD Potency Factor	1.208	1.04	0.68	0.1
Ammn (mg/l)	33.8	45.7	20	30
Time	Diurnal Factors			
09.00	1.304	1.344	2.04	1.67
10.00	1.219	1.045	1.54	1.48
11.00	1.159	1.01	1.19	1.38
12.00	1.123	0.848	1.07	1.2
13.00	0.777	0.964	0.94	1.08
14.00	0.72	0.726	0.86	1.03
15.00	0.994	0.749	0.9	1.01
16.00	0.944	0.795	0.94	1.01
17.00	1.152	0.717	1.00	1.1
18.00	1.164	1.145	1.07	1.12
19.00	1.174	1.322	1.09	1.13
20.00	1.027	1.011	1.06	1.14
21.00	0.915	0.936	1.08	1.09
22.00	0.836	1.103	1.11	1.04
23.00	0.961	0.996	1.22	1.09
00.00	0.716	0.817	1.17	1.1
01.00	0.329	0.645	0.99	0.89
02.00	0.232	0.227	0.61	0.64
03.00	0.163	no sample	0.41	0.48
04.00	0.171	no sample	0.25	0.4
05.00	0.317	0.539	0.26	0.34
06.00	0.672	0.701	0.43	0.44
07.00	1.146	1.156	0.76	0.64
08.00	1.282	1.333	1.81	1.29

It can be seen that significant differences exist between measured and default data sets when multiplying the Event Mean Concentrations (EMC) by the diurnal multipliers to obtain the actual DWF pollutant concentrations.

Chapter Eight

MOSQUITO Model Construction:- Storm Flows and Quality

8.1 Introduction

Sewer flow quality models are principally required to predict the pollutant loads discharged to WTPs and rivers in wet weather conditions. Consequently, the models also require to be calibrated using wet weather data. Storm modelling is more complex than DWF modelling as additional processes are introduced by the wet weather conditions e.g. sediment washoff from the catchment surface, gully pot inputs, and erosion of deposited in-sewer pipe sediments. This chapter describes how these additional processes were represented within MOSQUITO. The additional data which required to be collected is also discussed.

8.2 Sediment Washoff and Gully Pot Influence

The removal of pollutants from the catchment surfaces via surface washoff was represented by a modified form of the Price and Mance equations. There are three essential components to this model:-

- Erosion of sediments by rainfall impact.
- Erosion of sediments by flow (overland)
- Deposition of sediments from flow (overland)

Erosion by Rainfall Impact

This is defined by the following equation:-

$$E_i = a_i I^j \quad \text{eqn. 8.1}$$

where E_i = erosion rate of sediment mixture (kg/hr)
 a_i = calibration constant for surface type
 I = rainfall intensity (mm/hr)
 $j = 1.5$

Erosion by Flows (overland)

This is defined by the following equation:-

$$E_f = a_e (\tau - \tau_{ce}) \quad \text{eqn. 8.2}$$

where E_f = erosion rate of sediment fraction by overland flow (kg/hr)
 a_e = calibration constant for surface type
 τ = shear stress of the overland flow (N/m²).
 τ_{ce} = critical shear stress for erosion of sediment fraction (N/m²).

Deposition of sediments from flow (overland)

This is defined by the following equation:-

$$D_i = A_d (\tau_{cd} - \tau) \quad \text{eqn. 8.3}$$

where D_i = deposition rate of sediment fraction from overland flow (kg/hr).
 A_d = calibration constant for surface type.
 τ_{cd} = critical shear stress for deposition (N/m²).
 τ = shear stress of the overland flow (N/m²).

8.3 Simplified Model

- Generally differences are expected between pollutant concentrations entering gully pots and those leaving the gully pots, however the data used in the derivation of the default MOSQUITO surface washoff model showed no consistent differences between those pollutants entering and those leaving. Gully pot performance is therefore not modelled explicitly in MOSQUITO. The behaviour of the gully pots is accounted for by calibration from gully outflow data. Consequently the calibrated surface washoff model includes the effects of the gully pots.
- Of the three equations in the Price - Mance model only the first, *erosion by rainfall impact*, is considered important. This has been borne out from various research projects. The calibration constants for overland flow and erosion were therefore set to zero.

- MOSQUITO did not consider the build up of sediments on catchment surfaces between events. It is assumed that an unlimited amount of sediment is available for washoff (MOSQUITO User Manual, 1993).

Due to the conceptual nature of the gully pot model provided in MOSQUITO it was believed that no benefit would be gained via the utilisation of site specific data. Consequently, the default data proposed by the package were utilised as input data. (testing of the gully pot model was carried out to ascertain the sensitivity of the outputs to varying input quality characteristics, however the analysis showed that the results were not affected by changes to the input gully pot data. Consequently, the choice of which data set to use, site specific or default, proved to be an irrelevant question. This is discussed in more detail in chapter 9).

8.4 In-Sewer Processes

As deposited pipe sediment can be eroded by storm flows, additional data were required to describe the characteristics of these sediment. Approximately 20% of the Perth sewerage network is affected by sedimentation, therefore a significant investment was made to obtain data which could provide an accurate representation of the sediments characteristics.

8.5 Pipe Sediment Data Collection

Sediment data were collected for twelve different locations throughout the Perth catchment. Very substantial variations in sediment pollutant concentrations were observed from location to location. This is highlighted in tables 8.1 and 8.2.

Table 8.1 Spatial Variability of Pollutants Attached to Sediments in Pipes of Similar Gradient and Land-Use in Perth (approx. 1 mile sampling interval)

Sub-catchment:		Bridgend		
Determinand	COD (mg/l)	BOD (mg/l)	NH ₃ (mg/l)	Total Solids (mg/l)
Blended	10 100	1062	42	29 120
Mixed	5 500	787	32	10 560
Dissolved/Fine	2 600	375	36	2 850

Sub-catchment:		North Inch		
Determinand	COD (mg/l)	BOD (mg/l)	NH ₃ (mg/l)	Total Solids (mg/l)
Blended	47 200	7 798	177	64 670
Mixed	23 200	5 781	134	29 400
Dissolved/Fine	10 200	3 581	152	7 680

Table 8.2 Spatial Variability of Pollutants Attached to Sediments (close proximity - 250m)

Determinand Test	Site	COD (mg/l)	BOD (mg/l)	NH ₃ (mg/l)	Total solids (mg/l)
Blended	1	10 100	1 062	42	29 120
	2	16 000	3 197	50	27 910
	3	13 600	1 334	84	63 580
Mixed	1	5 500	787	32	10 560
	2	12 400	3 306	55	13 870
	3	3 000	417	66	8 330
Dissolved/fine	1	2 600	375	36	2 850
	2	5 800	1 931	52	4 450
	3	1 200	417	52	3 260

Site: 1 - Bridgend Sundial
Site 3 - Bridgend Graveyard

(28.7.93),
(23.7.93).

Site 2 - Bridgend Willowgate (3.8.93),

If pipe sediment erosion were to occur during either dry weather flow or storm conditions it is evident that substantially different pollutant release could result from location to location (**Ashley, 1993**). Consequently, it would be beneficial to represent the high degree of pollutant variability within the sediment characteristics file, ideally by defining unique sediment characteristics for each pipe in which sediment exists (**Jack et al, 1995**). Unfortunately this degree of flexibility was not possible within MOSQUITO as only global pipe sediment characteristics could be defined.

8.6 Summary

Storm flow quality modelling is more complex than modelling under dry weather conditions as more processes require to be considered. These processes are sediment washoff from catchment surfaces, gully pot contributions and erosion of deposited pipe sediment. Sediment data were collected to aid the calibration of the Perth MOSQUITO model, however, no surface sediment data were collected. This was because no benefit would have been gained due to the simplistic nature of the gully pot model used within the software package. The in-sewer pipe sediment data which were collected showed a substantial degree of variation in qualitative characteristics from site to site. Since this variation was so significant it was believed that unique sediment characteristics should be defined for each pipe in which sediment exists. The MOSQUITO model, unfortunately, did not have this capability. Consequently, a sensitivity test were carried out to ascertain the significance of this limitation. This test is detailed in chapter nine.

Chapter Nine

MOSQITO Sensitivity Testing

9.1 Introduction

Due to the substantial variation observed in the sediment pollutant characteristics, a sensitivity test was carried out to ascertain the sensitivity of output results with respect to different input pipe sediment pollutant data. The objectives of the tests were a) to determine how detailed pipe sediment characteristic surveys should generally be for sewer flow quality modelling and b) to shed light as to whether the simplification of defining *global* pipe sediment characteristics (physical and pollutant) was valid.

9.2 Sediment Pollutant Characteristics Sensitivity Test

The test carried out used the two data sets shown in table 9.1:-

Table 9.1 Pipe Sediment Characteristics

	Data A MOSQITO Default data	Data B Synthetic data
Shear Strength (N/m ²)	3	3
Wet Bulk Density (kg/m ³)	1688	1688
M.C. (%)	26	26
Fine Fraction (%)	5	5
Coarse Fraction (%)	95	95
Fine Fraction diam. (mm)	0.5	0.5
Coarse Fraction diam. (mm)	0.5	0.5
Fine Fraction density (kg/m ³)	1200	1200
Coarse Fract. Density (kg/m ³)	2650	2650
BOD Dissolved (mg/kg)	307	6000
BOD Fine Pot Fact	0.155	0.500
BOD Course Pot. Factor.	0.006	0.010

Test data A correspond to MOSQITO default data (**Gent et al, 1994**). Data set B has the same physical characteristics as data set A, but with substantially increased pollutant characteristics. Consequently the sediment eroded by any given event would be the same but a different pollutant release should occur.

Fig. 9.1 Rainfall and Flow Profiles

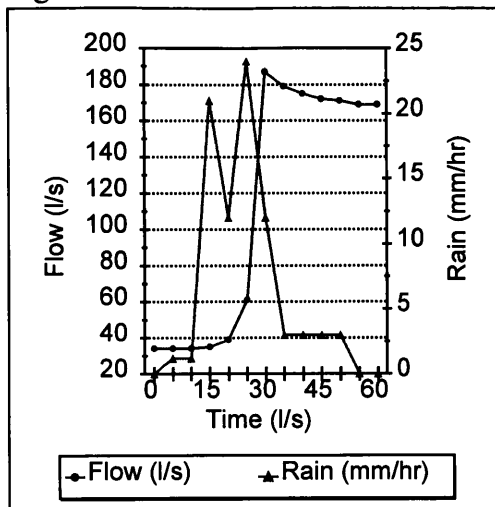


Fig 9.2 TSS and BOD (Data sets A and B)

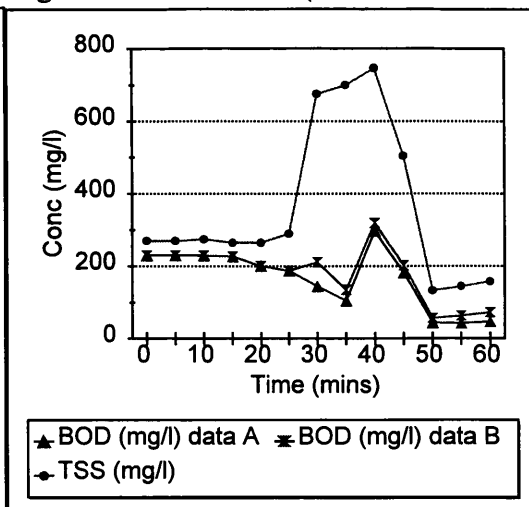


Fig.9.2 shows a TSS flush corresponding to the increased flow rate due to rainfall. However, no significant BOD flush can be observed. Comparing pipe sediment pollutant characteristics as shown in table 9.1, it would be expected that substantially different BOD profiles would result if pipe sediment erosion occurs, however both profiles remained virtually the same. From analysis of sediment depth data files, it was evident that deposited sediment had been eroded upstream of the gauge point and thus substantially different BOD profiles should have occurred. This however was not the case. The anomaly is attributable to MOSQUITO not releasing pollutants upon sediment erosion. Similar adjustments were made to the surface sediment pollutant characteristics however, the same results as shown above were obtained. This demonstrated the insensitivity of the models outputs to varying input data. Accordingly, if a pollutant flush was in fact observed in reality, the only way to represent this within the model would be to un-representatively increase the antecedent dry weather period. Such a procedure would constitute a force fitting of the model and furthermore, would create greater difficulties during the verification phase (Jack, 1995). These problems were discussed with Wallingford software, who accepted the comments but unfortunately were unable to offer any positive advice. It was therefore necessary to discard MOSQUITO as a useful modelling tool.

9.3 Summary

The objectives of the sensitivity test (which were firstly to ascertain how detailed the sediment data collection exercise should be and secondly to ascertain whether the sediment simplification utilised in MOSQUITO was valid) could not be established due to software problems. Consequently, MOSQUITO could not be used as the sewer flow quality model for this research project.

Chapter Ten

Hydroworks-PM/ Hydroworks-QM

10.1 Introduction

Hydroworks -QM is the latest U.K. sewer flow quality package and is compatible with the full solution hydraulic model Hydroworks-PM. As -PM is a full solution model the package provides a more suitable hydraulic base from which sewer flow quality calculations can be carried out. Consequently, the instability problem which occurred using WALLRUS would not be expected to occur with Hydroworks-PM. The Perth MOSQUITO model was therefore converted to Hydroworks -QM and a full evaluation of the package was carried out. Similar to the MOSQUITO evaluation, the objective of this work was to determine whether -QM could accurately represent the sewer flow quality for the City of Perth and therefore be used as the sewer flow quality modelling tool for the research. This chapter discusses the theory behind -QM and details the evaluation of the model.

10.2 Hydroworks -QM:- Background

Hydroworks -QM resulted from international collaboration between Wallingford Software Limited (UK) and Anjou Recherche (France). The package therefore benefits from the expertise gained through the development of MOSQUITO and the French sewer flow quality model, FLUPOL (Blanc et al, 1995; Phan et al, 1994).

10.3 Differences and Similarities (MOSQUITO & FLUPOL)

The fundamental difference between MOSQUITO and FLUPOL is that MOSQUITO is a deterministic model whereas FLUPOL is conceptual. Deterministic models use complex physical/biochemical laws to describe observed phenomena, conceptual models are much simpler, and use a more global representation of the observed phenomena. Generally this is done by defining transfer/calibration functions which convolute with input data to produce the necessary output data.

Although in this respect there is a large difference between MOSQUITO and FLUPOL there are however certain underlying similarities which exist between the two packages, that is - both treat separately the three basic processes which govern the quality of sewer flows:-

- Surface runoff and washoff following rainfall.
- Mixing of storm waters with foul flows, and transport through the sewerage system.
- Erosion and deposition of sediments within the sewerage system.

10.4 Surface Washoff Models (MOSQUITO & FLUPOL)

MOSQUITO uses the Price-Mance model as described in chapter eight. FLUPOL uses the linear reservoir model proposed by **Bujon, 1988**:-

$$Ms(Md/dt) = - KA Md \quad \text{Eqn. 10.1}$$

where

Md	mass of pollutants present on the ground (kg)
KA	coefficient depending on rainfall intensity
Ms	instantaneous mass of suspended solids (kg)

10.5 Sediment Transport Models (MOSQUITO & FLUPOL)

Ackers White equations (chapter seven) are used in MOSQUITO whereas in FLUPOL, Velikanov criteria are used:-

$$C_{min} = \eta_{min} \cdot \rho_s \cdot \rho_m \cdot (\rho_s - \rho)^{-1} \cdot U/w \cdot J \quad \text{eqn. 10.2}$$

$$C_{max} = \eta_{max} \cdot \rho_s \cdot \rho_m \cdot (\rho_s - \rho)^{-1} \cdot U/w \cdot J \quad \text{eqn. 10.3}$$

where

C_{min}, C_{max}	= limit sediment concentrations (g/l)
η_{min}, η_{max}	= efficiency coefficients
ρ_s	= density of sediment (kg/m ³)
ρ_m	= density of sediment and water (kg/m ³)
ρ	= density of water (kg/m ³)
U	= mean flow velocity through pipe section (m/s)
w	= sediment settling velocity (m/s)

J = energy line slope

'C' is the real concentration of the sediment. When 'C' is less than C_{min}, erosion will occur until 'C' becomes equal to C_{min}. When the concentration of the sediment lies between C_{min} and C_{max} then sediment will be transported at concentration 'C' without deposition or erosion. When the concentration of sediment is greater than C_{max}, deposition will occur until C = C_{max}.

10.6 Hydroworks -QM

It has been deduced from comparative studies that the washoff model used within FLUPOL provided better results than the Price-Mance model used within MOSQUITO (Blanc et al, 1995). Hydroworks-QM has therefore incorporated a conceptual washoff module. For the in-sewer sediment transport processes the Ackers White equations were retained.

10.7 Hydroworks -QM Washoff Model

This model is based on a single linear reservoir model. It is assumed that the pollutant flow at the outlet from the gully is proportional to the quantity of pollutants dissolved or suspended in the runoff.

$$Me(t) = L \times F(t) \quad \text{eqn. 10.4}$$

where

Me(t) = mass of pollutant dissolved or in suspension (kg)
 F(t) = pollutant flow (kg/s)
 L = linear reservoir coefficient.

The mass of pollution available is a function of the rainfall intensity and the mass of deposit on the ground.

$$dMe/dt = Kf \times Mr(t) - F(t) \quad \text{eqn. 10.5}$$

where

Mr(t) = mass of surface deposit pollution (kg)
 Kf = erosion/deposition factor related to rainfall intensity

The model is based on the behaviour of TSS and it is assumed that there is a proportional relationship between TSS and the other pollutants. The relationship is defined via potency factors.

$$F(t) = K_{pn} (i) \times F_m(t) \quad \text{eqn. 10.6}$$

where

$F(t)$	=	pollutant flow (kg/s)
K_{pn}	=	potency factor
$F_m(t)$	=	TSS flow (kg/s)

10.8 Initialisation

Background

The purpose of the antecedent period is to allow the readily erodible 'fine' sediments to build up on top of the in-erodible (-QM limitation), consolidated sediments (*the consolidated sediments are generally not in-erodible, but simply less erodible than the fine sediments. Nevertheless, due to limitations of knowledge surrounding the erosion and pollutant release mechanisms of the consolidated sediment layer these processes were removed from the -QM package. The implications of which are discussed in chapter twelve*). The longer the antecedent period, the greater the potential build up of these fine organic particles. The way -QM determines the concentration of the settled sediment however, is based on a steady state calculation. The model is run at the simulation start-time flow for the duration of the defined antecedent dry period. If settlement of suspended material occurs during this period then a flush can occur during the storm (if shear stresses are sufficient). If the steady state flow is too large and no settlement occurs then no flush can occur. The steady state initialisation approach is therefore very simplistic and gives no representation of the true 'build up' processes. This is because dry weather flows, in reality, do not remain constant, but vary with each hour of the day. Consequently organic matter which has settled from the flow at certain times of the day may be re-entrained into the flow at other times of the same day. The steady state Antecedent Dry Weather Period (ADWP) initialisation approach within QM could therefore be used as a simple calibration parameter. However this would not be good practice, as the ADWP is also used to define the build up of pollutants on the catchment surface and in the gully pots.

10.9 Conversion from MOSQUITO to Hydroworks -QM (DWF)

As the Ackers-White equations were retained within -QM, no conversion work was necessary for dry weather *quality* modelling. The only difference between the two packages was with respect to modelling of dry weather *flows*. In

MOSQUITO, flows were calculated via flow rates per area but in -QM flows are calculated using equations 10.7 and 10.8:-

$$\text{Base Flow} = \text{Per Capita Water Consumption} \times \text{Population} \times \text{Connectivity} \quad \text{eqn. 10.7}$$

$$\text{Hourly Flow} = \text{Base flow} \times \text{diurnal multiplier} \quad \text{eqn. 10.8}$$

where

connectivity = % of generated foul flow which drains to the sewerage system

As logger data provided values for base flows for each of the subcatchments, two unknowns were left in the first equation (given connectivity=100%). The first unknown was per capita water consumption, the second was population. The proposed default values within Hydroworks-QM were used for per capita water consumption. Population was then used as a calibration factor (table 10.1). The reason population was used as the calibration factor in preference to water consumption, was because -QM requires population to be defined as a 'population per contributing area', and not 'population per subcatchment'. It therefore requires to be demonstrated that population is not related to contributing area, as large impermeable areas may have little or no contributing population e.g. car parks drain a large volume of storm runoff, yet have little or no foul contribution. This suggests that population should be used as the calibration parameter as the accurate identification of actual population for modelling purposes is futile. It is important to note, however, that calibrating by population in preference to water consumption does not reduce the accuracy of the hydraulic calculations, however it does run against the grain of common practise. The corresponding dry weather flow plots are shown in figures D.5, D.9, D.13 and D.17.

Table 10.1 Land Use Characteristics

Land use	Location	Pop. Density (Pop/area)	Modelled Population.	Actual Population Est.	Water Consumption (l/h/d)
1	Muirton	138	8232	4500	190
2	Tullton	117	12690	7000	190
3	Rannoch	390	6423	6500	190
4	Bridgend	200	9420	2600	190
5	Craigie	187	27293	7500	190
6	Moncrieffe	74	2431	2900	190

Table 10.1 (Cont'd)

Land use	Location	Pop. Density (Pop/area)	Modelled Population.	Actual Population Est.	Water Consumption (l/h/d)
7	City Centre	80	1129	6000	190
8	Carrier pipes	0	0	0	0
9	Hillyland	65	3144	800	190

Table 10.1 demonstrates the significant difference between modelled population and actual population. This is a consequence of the problem detailed above (i.e the assumption that impermeable area and population are related. If per capita water consumption were used as the calibration parameter in preference to water consumption, very unusual values of water usage would also have resulted. This could mislead practising engineers to think of infiltration problems. Population is therefore considered to be the more appropriate calibration parameter).

10.10 Model Verification - Dry Weather Quality

The default fine sediment characteristics within -QM are diameter 0.05mm and specific gravity 1.7. These characteristics did not however model the transportation of the suspended sediment throughout the Perth sewerage system adequately. More suitable profiles were obtained when the sediment diameter was increased to 0.15mm and specific gravity decreased to 1.3. (Figs. D.7, D.11, D.15 and D.18). Figure D.18 shows that the modelling of TSS at the system outfall. The modelled error with respect to the TSS load passing the sampling point over the twenty four hour period was calculated as only -12%. The maximum error occurred at hour 24 of the simulation and was +900%. A peak sampled concentration of 515mg/l occurred at 10am. The modelled peak was observed to be 505mg/l at 9am (1 hour time lag). Consequently, the error with respect to the peak was -2%.

The modelled error with respect to the total Ammonia load passing the sampling point over the twenty four hour period was calculated as +37%. Figure D.19 shows that the largest ammonia error (+103%) occurred at hour 18. An observed peak concentration of 36mg/l occurred at hour 1. As no time lag was believed to be present the modelled concentration was 25mg/l. This produced an error -30%.

With respect to COD at the system outfall, it can be seen from figure D.20 that the largest errors occur at hours 9 and 24. These errors were +96% and +166% respectively. The modelled error with respect to the total COD load passing the sampling point over the twenty four hour period was however calculated as only

+26%. An observed peak concentration of 718mg/l occurred at hour 8. The modelled peak occurred one hour later and was 863mg/l. This produced an error of +20%.

Table 10.2 Sediment Calibration Parameters

Model	Diameter	Specific Gravity
MOSQUITO (Calibrated Model)	0.5mm	1.010
Hydroworks -QM (Calibrated Model)	0.15mm	1.3

Table 10.2 highlights the difference between the sediment characteristics required to calibrate the -QM and MOSQUITO models. The large difference, at first may appear strange, as the same sediment transport equations (Ackers-White) are used in both models. It would therefore be expected that similar sediment characteristics would be required to calibrate both -QM and MOSQUITO. The reason for these large differences is however discussed in section 10.111.

10.11 COMPARISON OF ACCURACY - QM & MOSQUITO (DWQ)

In order to ascertain the integrity of the converted model a comparison was made between the accuracy of the two packages. This comparison was carried out at the system outfall (Friarton pumping station).

Table 10.3 Accuracy of Model Predictions:- MOSQUITO & -QM

ACCURACY OF PREDICTION (total load passing sampling point over twenty four hour period)			
	TSS	COD	AMMN
MOSQUITO	-8%	+30%	+17%
Hydroworks-QM	-12%	+22%	+32%

As can be seen from table 10.3, -QM is in general, no less accurate than the original MOSQUITO model. The modelling of COD has been improved by 8%, although the accuracy of TSS and Ammonia was reduced by 4% and 15%, respectively. The modelling of ammonia proved to be the most problematic, as the accuracy of prediction could not be improved by subtle calibration. It was thought possible that the larger -QM errors (ammonia) may have been a result of the model ignoring the dilution effects of infiltration. These flows were taken into consideration during the calibration of the MOSQUITO model but ignored

during calibration of -QM. The reason for this omission in the latter modelling is explained below.

10.111 Infiltration Modelling

The total flow in a sewer pipe is a mixture of both foul and infiltration flow, with the quantity of infiltration flow being independent of, and less polluted than the foul component. Consequently, greater accuracy could (theoretically) be gained during simulation if the total flow was considered as two separate flow types. In order to test this hypothesis the infiltration flows within the Perth system were considered in the MOSQUITO model and ignored in the -QM model (as discussed above). Consequently the sediment characteristics as shown in table 10.2 correspond to the characteristics required to calibrate the Perth sewer quality model with (MOSQUITO) and without (-QM) consideration being given to infiltration. Table 10.2 shows the significance infiltration had upon the calibration parameters.

10.112 Infiltration Modelling Procedure

The procedure adopted for the modelling of infiltration in MOSQUITO was to estimate how much of the flow at the sampling points (heads of subcatchments) was made up of foul sewage and how much was made up of infiltration. Once this had been defined the respective flows were input into the MOSQUITO model as two separate flow types. The procedure adopted for inputting this infiltration into the model was to estimate the quantity of infiltration present at the subcatchment outfalls (data obtained from the data collection exercise). The differences between the quantity of infiltration at the top and bottom of the subcatchments were then proportionally introduced between the respective pipes. A qualitative assumption was made that the infiltration flows were clean and that all the pollutants analysed in the laboratory were associated with the 'foul only' part of the flow. This necessitated new EMCs to be input into the model and new diurnal pollutant profiles calculated.

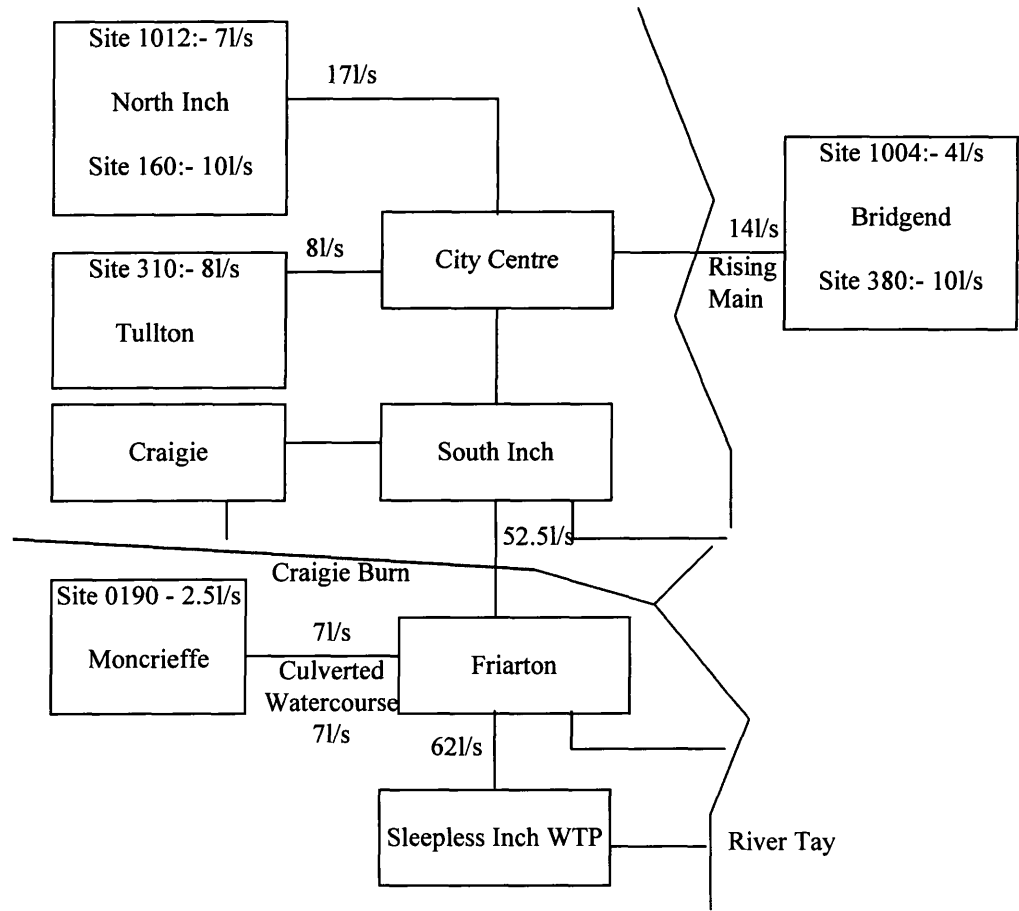
10.12 Identification of Infiltration for the Perth System

(To be read in conjunction with figure 10.1 overleaf)

Infiltration at site 1004 (Bridgend) was based on the measured average value of 4l/s obtained at site 1004 during the data collection exercise. Infiltration at site 380 was measured to be 10l/s thus giving 14l/s of infiltration flowing through the rising main. Infiltration at logger sites 1012 (North Muirton), 0310 (Tullton), and 0160 (North Inch) was estimated to be 7l/s, 8l/s and 10l/s respectively.

The City Centre and the main pipes coming down the catchment before the Craigie connection were based on the infiltration flow arriving at the North Inch - site 0160. A value of 52l/s was therefore obtained below the Craigie connection (above overflow 15 - South Inch P/S). It was then estimated that 7.5l/s of infiltration was coming from the culverted watercourse (site 1009), giving a total of 59.5l/s. At site 0190 (Moncrieffe) the infiltration was established as 2.5l/s which then gave a cumulative infiltration value of approximately 62l/s for the whole catchment. This figure is believed to be realistic due to the permeability of the granular soil and the locality of the river.

Fig. 10.1 Infiltration Assessment of the Perth Drainage System



10.13 -QM Infiltration Modelling Analysis

In order to ensure the differences in sediment characteristics (as shown in table 10.2) were not biased by any differences within the -QM and MOSQUITO packages, infiltration flows were also modelled within -QM. This work showed that very similar sediment characteristics were required to calibrate -QM and MOSQUITO when infiltration was considered in both models. This is demonstrated in table 10.4 below.

Table 10.4 Sediment Calibration Parameter Comparison

Model	Diameter	Specific Gravity
MOSQUITO (Infiltration)	0.5mm	1.010
Hydroworks -QM (Infiltration)	0.5mm	1.035
Hydroworks -QM (No infiltration)	0.15mm	1.3

Table 10.4 demonstrates that the variations in sediment characteristics as shown in table 10.2 were due to infiltration and not due to differences within the respective packages.

Figures 10.2 to 10.5 compare the pollutographs as produced by the -QM infiltration and the -QM non-infiltration model. Table 10.5 compares the modelled errors at the system outfall for the ‘infiltration’ and ‘no infiltration’ models. It can be seen that both models are calibrated to a similar degree of accuracy therefore demonstrating the validity of the comparison made in table 10.4.

Table 10.5 Infiltration and No Infiltration Model Comparison

ACCURACY OF PREDICTION (total load passing sampling point over twenty four hour period)			
	TSS	COD	AMMN
Infiltration Model	+8%	+24%	+35%
No Infiltration Model	-12%	+22%	+32%

Fig. 10.2 -QM Infiltration Comparison at System Outfall - Flow

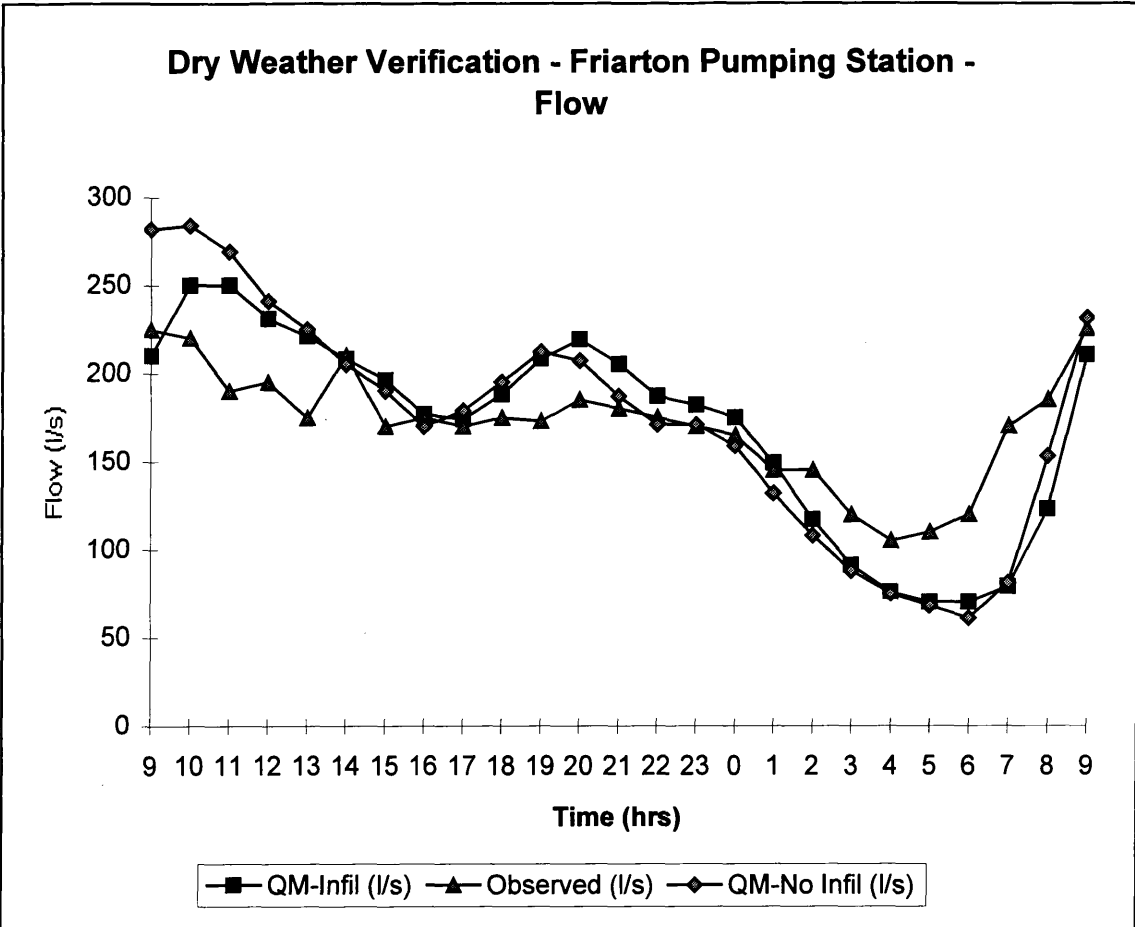


Fig. 10.3 -QM Infiltration Comparison at System Outfall - Ammonia

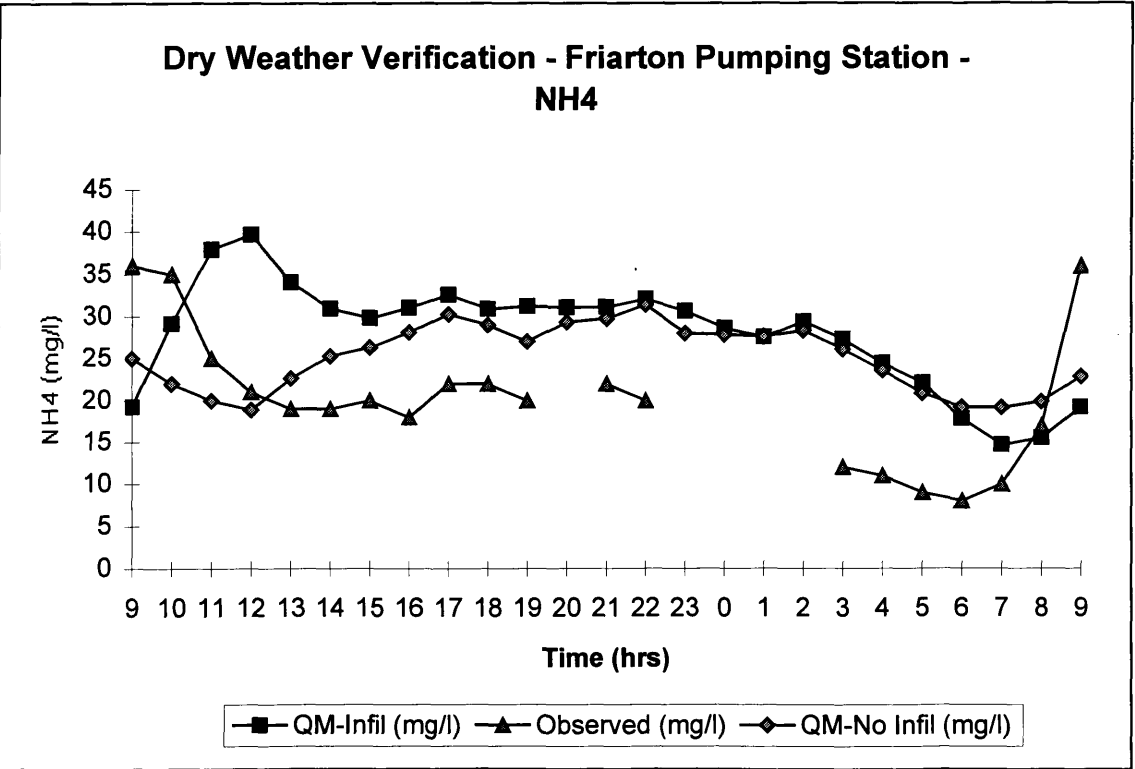


Fig. 10.4 -QM Infiltration Comparison at System Outfall - TSS

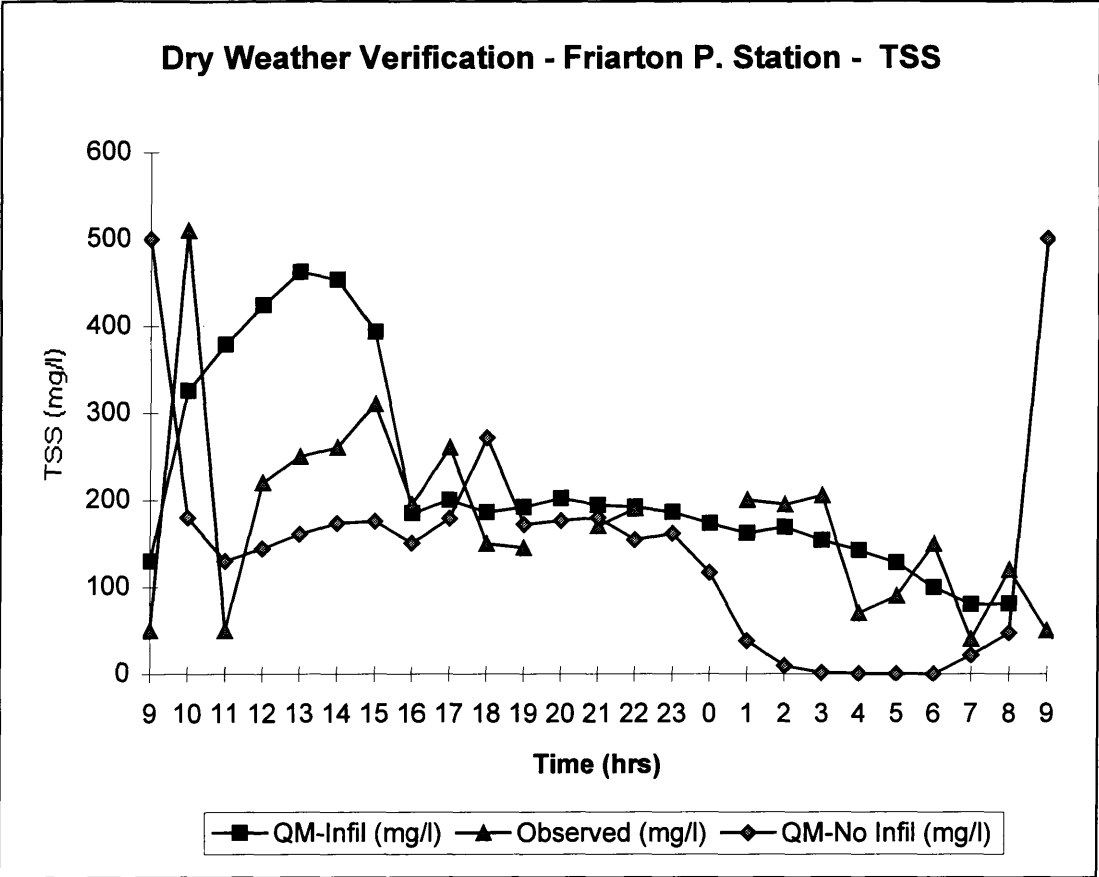
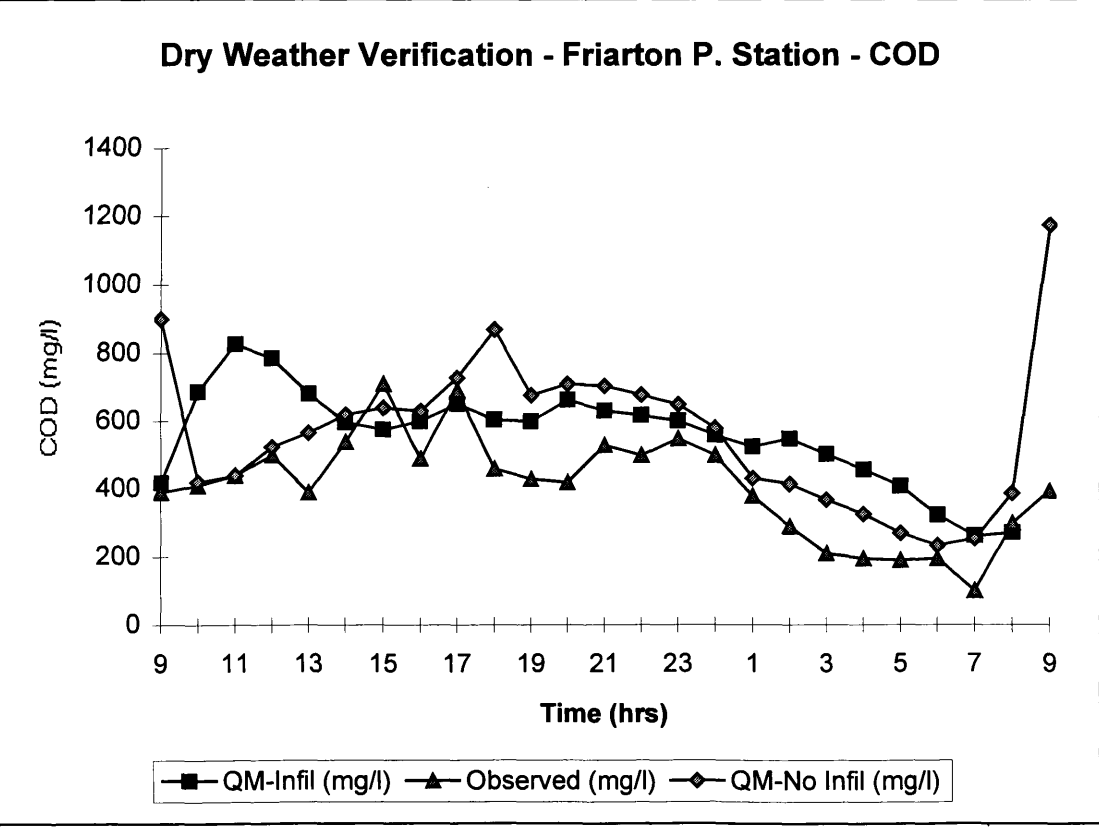


Fig. 10.5 -QM Infiltration Comparison at System Outfall - COD



10.14 Discussion

As can be seen from table 10.5 and figures 10.2-10.5, the infiltration model has represented flows and pollutants to a degree of accuracy which is in general no greater or less than the original -QM 'no infiltration' model (which did not explicitly consider infiltration). The reason the infiltration model did not prove inferior under dry weather conditions was because the sediment characteristics were finely adjusted until the determinands were modelled acceptably. As a consequence different sediment characteristics were obtained for the respective models. This therefore meant that the 'infiltration' and 'no-infiltration' models could theoretically produce different pollutographs for any given rainfall event. This was investigated via sensitivity analysis and is discussed in chapter twelve.

10.15 Ammonia Modelling

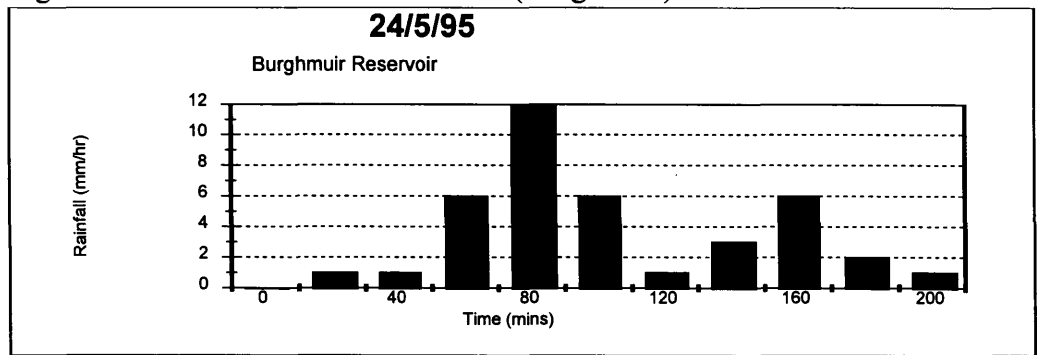
It was initially thought that the original model's over prediction of ammonia may be solved via the consideration of infiltration. This however proved not to be the case as infiltration did not improve these fits (fig. 10.3 and table 10.5). The poor modelling has therefore been attributed to a 'glitch' in the software. During 'beta-testing' of the software it was found that ammonia required to be modelled under the heading of Total Kjeldahl Nitrogen (TKN). The reason for this was that a mistake had been made during the writing of the programme code. It was the 'opinion' of Wallingford Software however that this mistake was purely aesthetic and that although ammonia was being modelled under the heading of TKN it was ammonia calculations which were being carried out, and not TKN calculations. Nevertheless, the possibility that -QM was calculating TKN instead of ammonia was not ruled out. This, still did not fully explain the over prediction, because no matter which determinand was being modelled -QM simply routes the pollutant through the system. Consequently if a determinand is being modelled correctly at the heads of the subcatchments, the output data should also remain accurate, if inputs are correct. It was therefore thought that an ammonification process may have been *inexplicitly* accounted for within -QM (-QM gives no *explicit* consideration of water quality processes) thus increasing the ammonia content of the sewage as it travels through the system (ammonification is the process where heterotrophic bacteria breakdown the proteins held within organic nitrogen, producing ammonia). Wallingford Software stated however that the model does not explicitly or inexplicitly account for any biological processes occurring within the sewerage system (Sanderson, 1996). Consequently, the reason

for the over prediction was not fully known or understood and could only be speculated.

10.16 Model Construction & Verification - Storm Flows

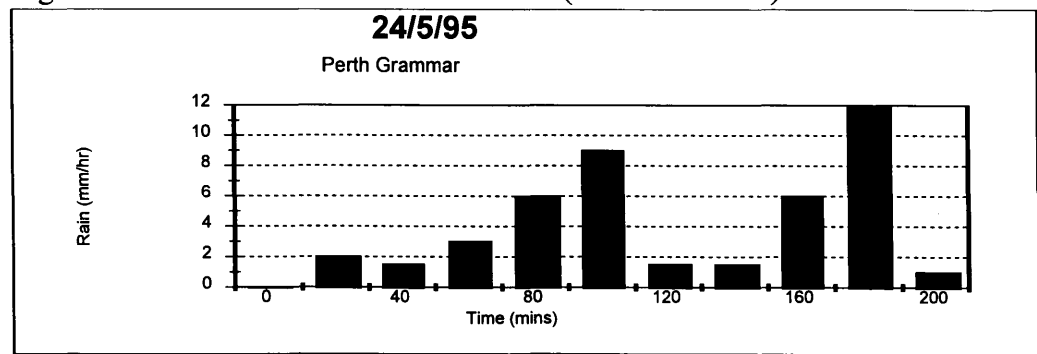
Within the period of time passed since the completion of the original hydraulic model (WALLRUS), sewerage maintenance and relining work had taken place. These changes therefore required to be accounted for within the model. An additional, limited survey, was therefore carried out to provide new sediment depths and pipe roughness data. It was concluded that only minor changes to the model required to be made (**Fraser, 1995**). Nevertheless, the accurate identification of sediment depth and pipe roughness is vital for sediment transport and thus sewer flow quality modelling, as both affect in-sewer flow velocities. Various rainfall events were used to ascertain the integrity of the updated model. These events, and the corresponding flows, can be seen in figs. 10.6a -10.7g

Fig. 10.6a Rainfall Profile for 24/5/95 (Burghmuir)



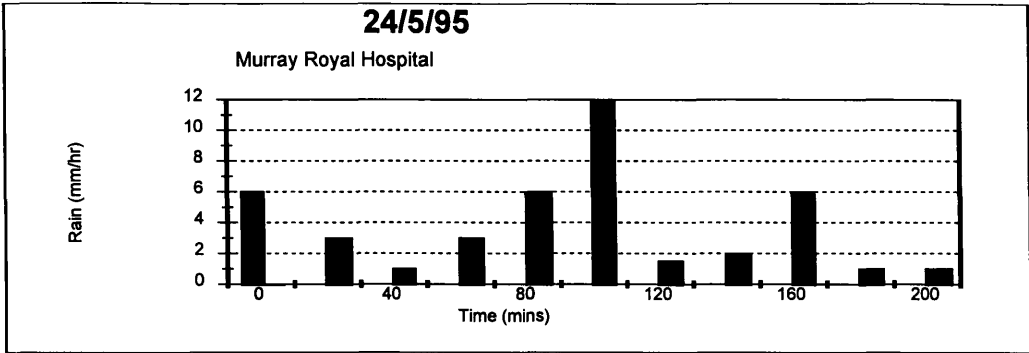
(Time measured from 20:00)

Fig. 10.6b Rainfall Profile for 24/5/95 (Perth Grammar)



(Time measured from 20:00)

Fig. 10.6c Rainfall Profile for 24/5/95 (Murray Royal)



(Time measured from 20:00)

Fig. 10.6d Modelled and Observed Flow – Bridgend (24/5/95)

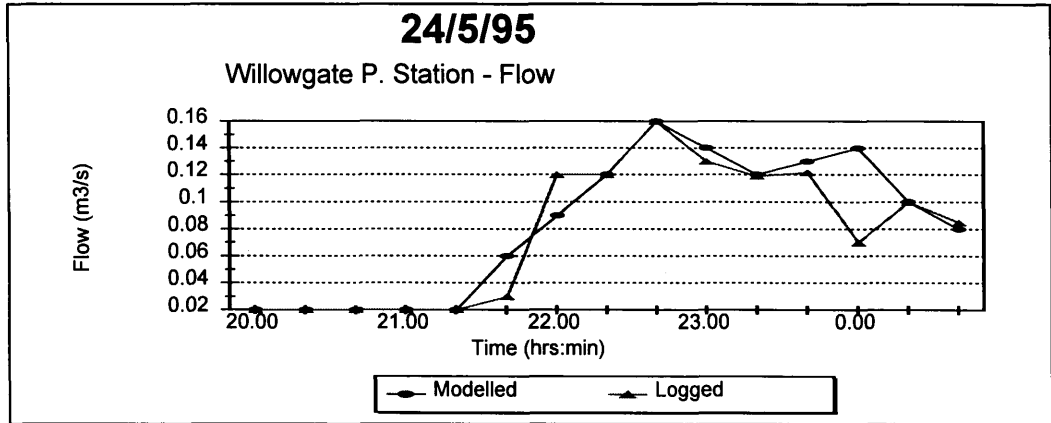


Fig. 10.6e Modelled and Observed Velocity – Bridgend (24/5/95)

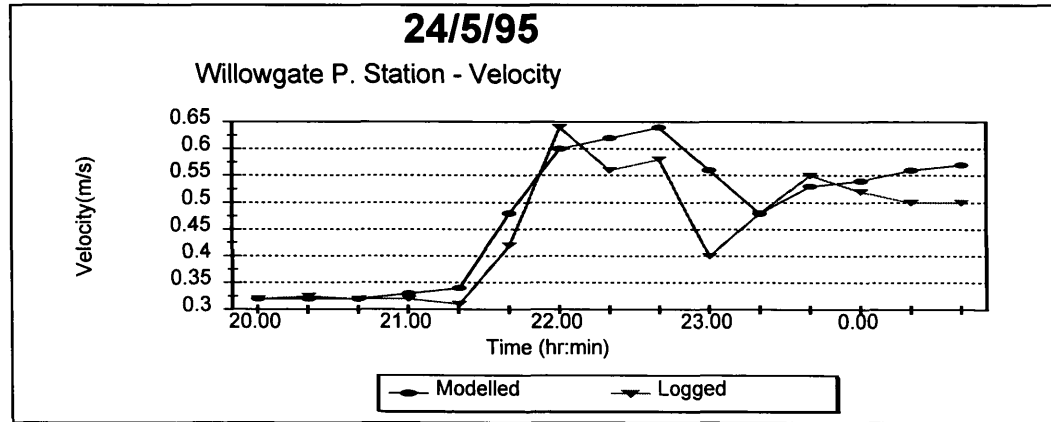


Fig. 10.6f Modelled and Observed Flow – Craigie (24/5/95)

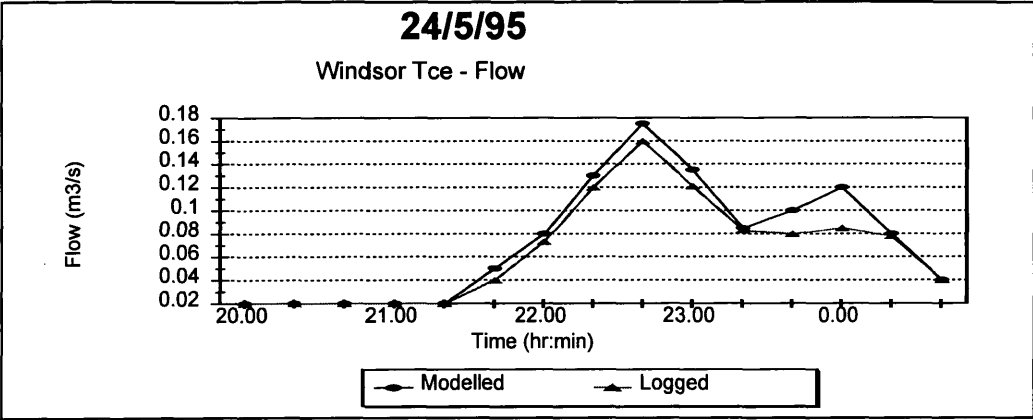
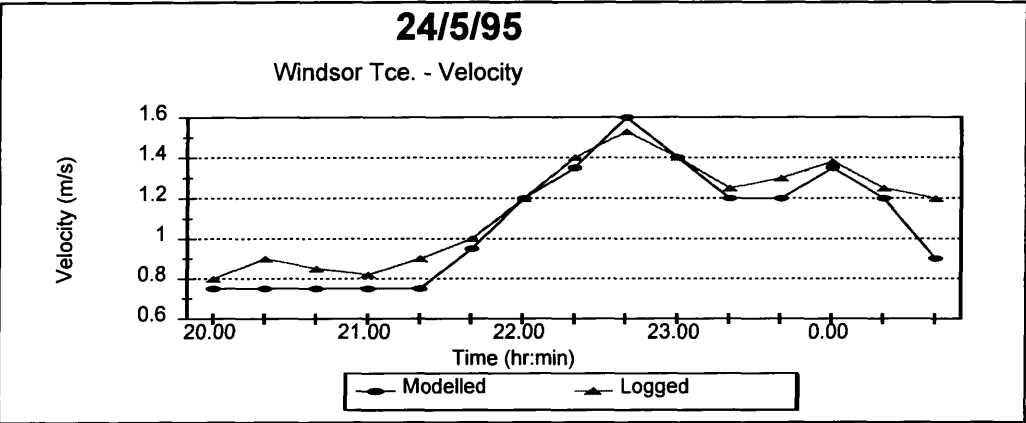
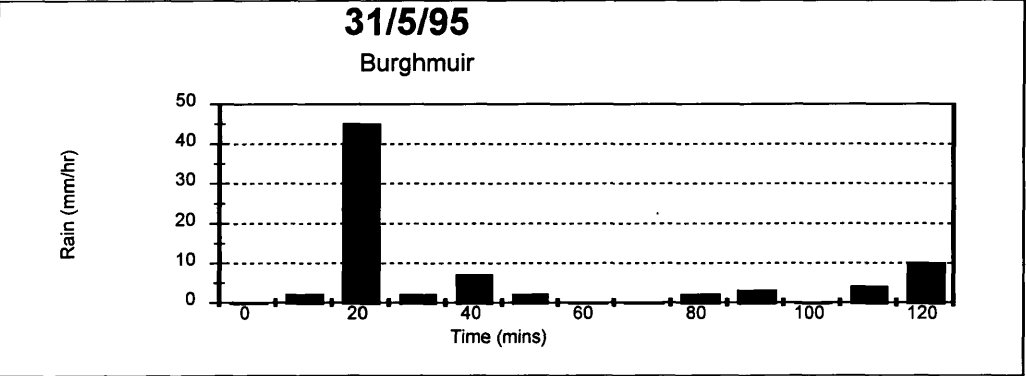


Fig. 10.6g Modelled and Observed Velocity – Craigie (24/5/95)



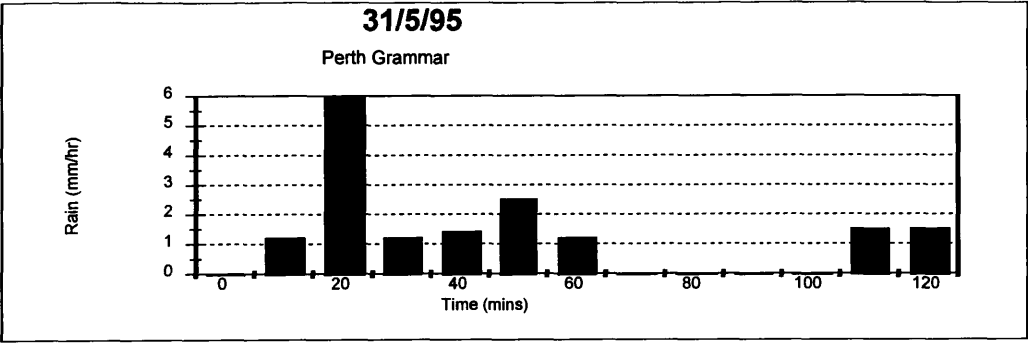
Rainfall Event 31/5/95

Fig. 10.7a Rainfall Profile for 31/5/95 (Burghmuir)



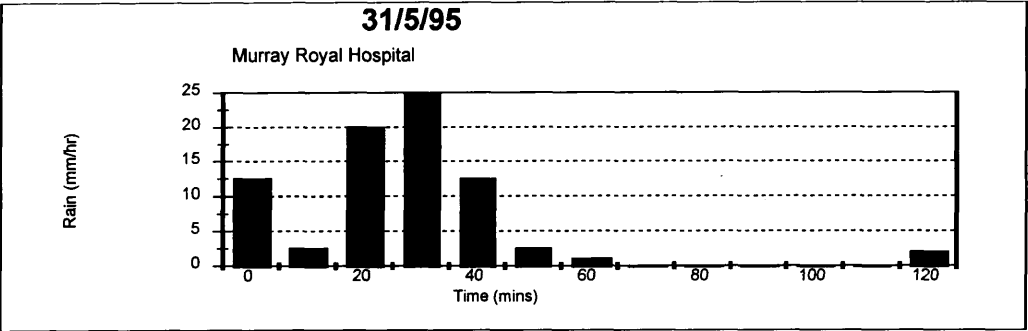
(Time measured from 10:30)

Fig. 10.7b Rainfall Profile for 31/5/95 (Perth Grammar)



(Time measured from 10:30)

Fig.10.7c Rainfall Profile for 31/5/95 (Murray Royal)



(Time measured from 10:30)

Fig. 10.7d Modelled and Observed Velocity – Bridgend (31/5/95)

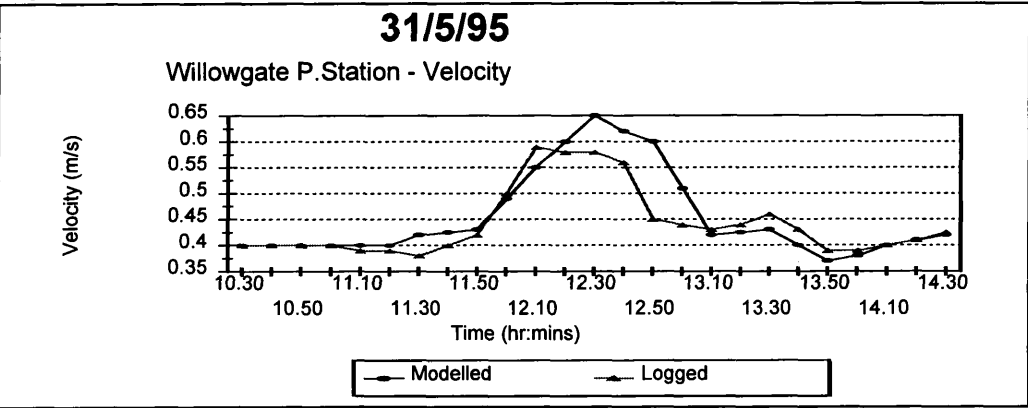


Fig. 10.7e Modelled and Observed Flow – Bridgend (31/5/95)

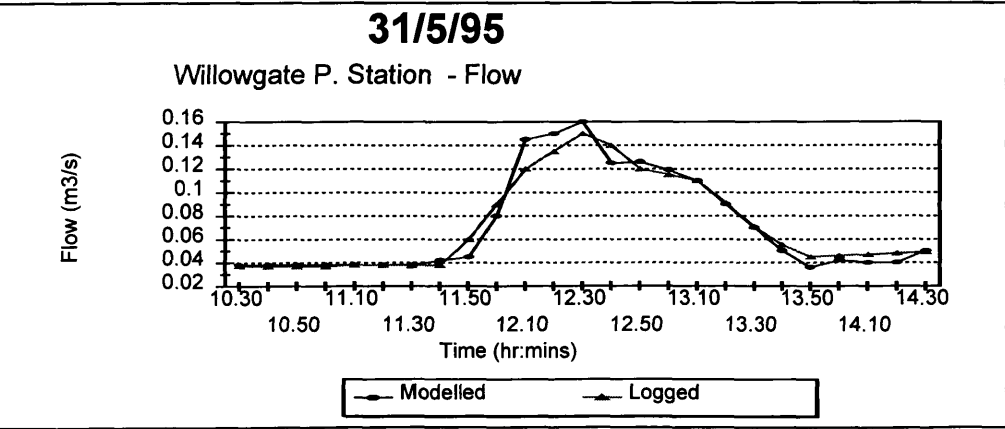


Fig. 10.7f Modelled and Observed Flow – Craigie (31/5/95)

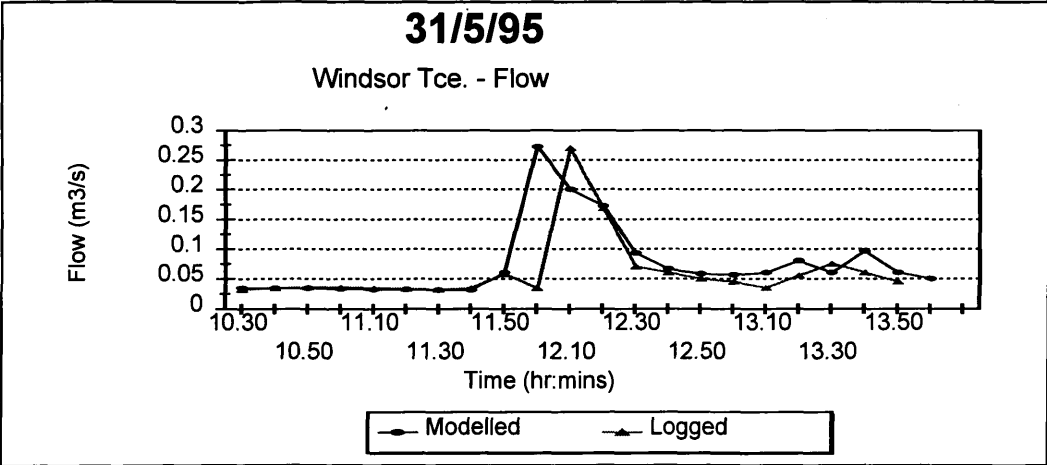
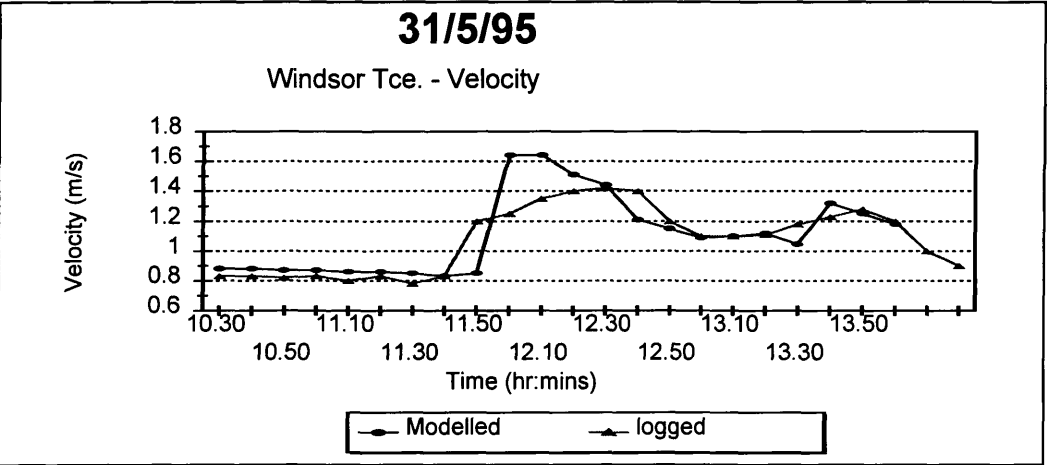


Fig. 10.7g Modelled and Observed Velocity – Craigie (31/5/95)



10.17 Conclusions (Hydraulic model)

Table 10.6 provides a summary of the modelling errors which were obtained in the re-verification exercise.

Table 10.6 Hydraulic Modelling Errors (Volume and Peak Flows)

	Willowgate P.S. (Bridgend)	Windsor Tce. (Craigie)
Modelled Error:- Volume (24/5/95)	+8%	+14%
Modelled Error:- Peak (24/5/95)	0%	+11%
Modelled Error:- Volume (31/5/95)	-2 %	+22%
Modelled Error:- Peak (31/5/95)	+6.5%	+3%

The **WAPUG Code of Practise for Hydraulic Modelling, (1998)** defines tolerable errors of between +25% to -15% for peak flows and +20% to -10% for volume. As the errors shown in table 10.6 are generally within these limits the model was considered to be suitably verified. Consequently, no additional adjustments/calibrations were made to the model.

10.18 Quality Model Construction & Verification - Storms

The storm quality model construction and verification for COD, TSS and Ammonia is detailed in Appendix B. Although the problems as experienced with MOSQUITO did not occur using -QM, Appendix B shows that significant problems were still encountered.

Chapter Eleven

BOD Modelling

11.1 Introduction

The analysis, as detailed in Appendix B, was concerned with the modelling of TSS, COD and Ammonia. No consideration was given to BOD. This was not because BOD was omitted in the calibration/verification process, but because for clarity, it was thought necessary to discuss the modelling of BOD and COD separately. The reason for this was that dry weather BOD concentrations were sampled and analysed only at every fourth hour of the twenty-four hour data sets. This made it difficult to determine whether BOD was being modelled to a suitable degree of accuracy under DWF conditions, as certain peaks or troughs in sampled concentrations may have been missed. As both COD and BOD have a sediment attached component, it was thought logical to assess the package, initially, using TSS and COD as the greater amount of data would allow problems to be noted more readily.

11.2 BOD - Dry Weather

The BOD profiles, for various sub-catchments, are shown in figures D.21-D.23. Figure D.24 shows the goodness of fit at the system outfall (Friarton Pumping Station). In order to calculate the modelled error with respect to total BOD load, significant interpolation of the observed data would have been required. This was because the BOD data were collected at only four hourly intervals. No attempt was therefore made to determine the modelled error with respect to the BOD load under DWF conditions as this analysis would have been meaningless. However, with respect to the peak, it can be seen from figure D.24 that an error of +250% occurred at 17:00hrs at the system outfall.

11.3 Storm Verification (BOD)

11.31 Rainfall Event 24/8/95

Figure D.25, compares modelled and observed flows in the Bridgend sub-catchment for the 24/8 event. Figure D.26 illustrates that although correlation between modelled and sampled BOD data was poor, the general trend of the BOD profile at the Willowgate Pumping Station was represented. Interpolation of the observed BOD data suggested that -QM over-predicted the total load by 47%. The peak sampled concentration occurred just before 10.00am and was 322mg/l. -QM over-predicted this peak by 43%. However, the maximum error with respect to concentration occurred at 10.45am and was +218%. A possible cause for the general over prediction may be due to the base dry weather model over predicting BOD throughout the duration of the rainfall event. An additional DWF data collection exercise was intended to be carried out to provide more information concerning this anomaly, however due to time constraints it was not undertaken. It should however be noted that although BOD is over predicted, the total TSS load was under predicted by -47% (fig. B.1f). This may be related to inadequate potency factors. However, in order to bring modelled BOD down to observed values, very substantial calibrations would have to be made. Consequently, the DWF model would require re-calibration. This degree of parameter adjustment was not justifiable as the potency factors were defined from laboratory data.

The modelled and observed flow profiles at the South Inch Outfall for the 24/8/95 event are shown in figure D.27. With reference to figure D.26 and figure D.28, it can be seen that BOD has been modelled to a higher degree of accuracy at the South Inch Pumping Station as the total BOD load error was only -12%. The modelled error with respect to the peak sampled BOD concentration was -16%. The maximum modelled error occurred at 10.45am and was -19%. It can be seen that the modelled BOD profile is generally low at the South Inch Pumping Station, whereas in Bridgend, figure D.26, it is generally high. Consequently,

decreasing the BOD potency factor or increasing the settleability of the sediment to improve the accuracy of the BOD predictions in the Bridgend sub-catchment would result in a loss of overall accuracy (TSS and BOD) at the systems outfall. This was therefore not carried out as the South Inch Pumping Station is a site of greater strategic importance than the Bridgend site, as it is one of the main overflows in the Perth system.

11.32 Rainfall Event 29/8/95

With reference to figures D.27 and D.29 it can be seen that the 29/8/95 event was of a similar magnitude to the 24/8/95 event, and occurred at the same time of day. Consequently, it would be expected that the modelling accuracy for the two events would be similar. However, it can be seen from figs. B.1f, B.1k, D.28 and D.30 that -QM is generally more accurate for the 24/8/95 event. The modelled error with respect to total BOD load for the 29/8/95 event was -40%, whereas, for the 24/8 event the total load error was only -12%. The difference in accuracy is most likely due to the temporal variability of sewer flow quality. Consequently, varying degrees of modelling accuracy may be expected for different albeit, similar rainfall events.

11.33 Rainfall Event 31/5/95

The 31/5 event, which was of high intensity, showed a poor representation of TSS in the Bridgend sub-catchment (fig. B.3f), although BOD was modelled surprisingly well (fig. D.32). The modelled error with respect to the total TSS load was -82 %, whereas the modelled error with respect to the total BOD load was only -13%. It can be seen from figure D.32. that the sampled peak concentration occurred at 11.52am and was 304mg/l. -QM under predicted this peak by 26%. The maximum error occurred at 12.08pm and was +34%.

The modelled error with respect to the total BOD load in the Craigie sub catchment was +73%. The maximum error with respect to concentration occurred at 11.52am and was +104% (fig. D.34. The peak BOD concentration was

sampled at 11.36 and was 192mg/l. -QM over predicted this peak by 23%. The modelled flows at this location can be seen in figure D.33

It would appear that -QM modelled the concentrations of BOD more accurately in Bridgend, than in Craigie, yet it is the Bridgend subcatchment which is affected by sedimentation.

Figure D.35 compares modelled and observed BOD data at the system outfall (South Inch Pumping Station). As no flow data were available the total load error could not be calculated. However, the average and peak errors with respect to BOD concentration were +31% and 10.5%, respectively.

11.4 BOD Modelling Conclusions

For the large intensity rainfall event (31/5/95) -QM modelled BOD with a surprisingly good degree of accuracy. For the lower intensity events (<2mm/hr) the accuracy was less obvious. Although it should be borne in mind, however, that the consequences of poor modelling under low intensity events are less significant. Table 11.1 provides a comparison of the BOD and COD modelling errors for the various events. Bold denotes better modelling.

Table 11.1 Comparison of BOD and COD Storm Modelling Errors

Modelled Errors	24/8/95	29/8/95	31/5/95
Load			
Bridgend (BOD:COD)	+47% : -8%	No comparison possible	-13% : -48%
Craigie (BOD:COD)	no comparison possible	no comparison possible	+73% : +75%
South Inch (BOD:COD)	-12% : +12%	-40%: +5%	no comparison possible

It can be seen from table 11.1 that BOD was not modelled to a higher level of accuracy than COD, or vice versa. Consequently, it can be concluded that

problems are apparent with the qualitative representation of both determinands. Nevertheless, the analysis showed that the principal problem still laid with the representation of TSS for the 31/5 event. This is discussed further in chapter twelve.

Chapter Twelve

Hydroworks -QM Sensitivity Testing

12.1 Introduction

Since the QM model grossly underestimated the TSS pollutant profiles for the largest rainfall event (31/5/95), it was necessary to ascertain whether the modelling inaccuracy was solely due to -QM not considering the potential pollutant release from deposited sediment erosion, or whether the model simply required further calibration. The purpose of these sensitivity tests were therefore not purely to determine the sensitivity of -QM to key parameter changes, but to ascertain whether the modelled outputs could be improved by sensible calibration. The 31/5 rainfall event was chosen for the sensitivity tests as it was this rainfall event which caused the greatest modelling inaccuracies. Emphasis was placed upon TSS and COD as these determinands were the most poorly modelled. Sensitivity testing was also carried out using the infiltration model. The objective of this analysis was to ascertain whether this model would produce different/better results than the model which did not consider infiltration. This chapter details these tests and draws upon the results to determine whether -QM was suitable for use within this research project.

12.2 Sensitivity Test Strategy

12.21 Sensitivity Test 1A

During simulations an initialising period was required before the qualitative calculations reached an 'equilibrium'. The purpose of the first tests was to ascertain the minimum duration the simulation should be started prior to the beginning of the rainfall event.

12.22 Sensitivity Test 1B

The sediment/pollutant build up, which accumulates on top of the consolidated pipe sediment prior to a rainfall event is calculated using a steady state calculation. This calculation uses the dry weather flow and pollutant concentration which corresponds to the simulation start time. These conditions are run through the model for the duration of the ADWP. If the flow rate at the simulation start time is of sufficient magnitude, no deposition will occur, whereas if the flow rate is low then particles may settle out of the flow and build up on the sediment bed. When flow rates increase, corresponding to the onset of storm conditions, this easily erodible deposited sediment can be re-entrained into the flow thus representing a flush. However, as the simulation requires to be started prior to the beginning of the rainfall event (as discussed in test 1A), then theoretically different pollutant build-up could result, depending on whichever simulation start time is utilised. The reason for this is because flow rate (and pollutant concentration) and thus transport capacity will be different for each different simulation start time. Sensitivity analysis was therefore carried out to ascertain the affect simulation start time has on the modelled outputs during the storm.

12.23 Sensitivity Test 2

The original Hydroworks -QM model required different sediment characteristics to produce a verified DWF model than the model which considered infiltration. Consequently a sensitivity test was carried out to determine whether the two models would produce different pollutographs for any given rainfall event.

12.24 Sensitivity Test 3

As shown in Appendix B (fig. B.3f), the modelled suspended solids profiles at the Willowgate Pumping Station (Bridgend) for the 31/5/95 were extremely low compared to the observed values. Sensitivity tests were therefore carried out to determine whether this was purely due to the package not modelling the erosion of consolidated pipe sediments, or whether the sediment characteristics simply required further calibration. Consequently sediment diameter and specific gravity

were altered to the maximum/minimum values which would still produce a calibrated dry weather flow model. The storm event was then re-run in order to ascertain if modelled outputs would improve.

12.3 Sensitivity Tests:- Discussion and Results

12.31 Test 1A (Initialisation Period for Quality Calculations)

Sensitivity testing was carried out on ADWP because qualitative inaccuracies were found to occur at the beginning of simulations. Consequently, to prevent these inaccuracies from affecting the model results during periods of interest (i.e. during storms) the simulations were started earlier than the start time of the rainfall event. This allowed the quality model time to reach a state of 'equilibrium' before the relevant calculations were carried out. However, if the simulation was started at say 9am with an ADWP of 50hrs then a different flush could result in comparison with another simulation which was started at 7am with a ADWP of 48hrs. This is because the flow rates at 7am and at 9am are different, and thus different transport capacities would exist. Sensitivity testing was therefore carried out to ascertain the significance, if any, of starting a simulation prior to the start time of the rainfall event.

Test Details

Simulation 1:-	Simulation Start Time 10:30 ADWP:- 26 hrs
Simulation 2:-	Simulation Start Time 08:30 ADWP:- 26 hrs

{Note:- the start time of the rainfall was 11:30, and the actual ADWP was 26 hrs}

The results (figure D.36) show that by starting the simulation at 8.30am the qualitative calculations are properly initialised before the rainfall event begins (11.30am). For the simulation which was started at 10.30am, the quality calculations are still initialising at the beginning of the event. The conclusions from this test were that starting the simulation prior to the rainfall event is important for overall accuracy and that the simulation should be started not less than 3hrs before the beginning of the rainfall event.

12.4 Test 1B - Background, Discussion and Results

The previous test was concerned with ascertaining the sensitivity of the model with respect to different start times, albeit with similar ADWP. However, more realistic simulations could 'theoretically' be obtained by adjusting the ADWP to account for the earlier simulation start time. The sensitivity of the models outputs was therefore investigated using the following data:-

Test Data

Simulation 1	start time 08.30	ADWP 26hrs
Simulation 2	start time 08.30	ADWP 23hrs
{Note:- Rainfall at 11:30, ADWP:- 26 hrs}		

As the rainfall event occurred at 11.30am, and had an ADWP of 26hrs then simulation no.2 defined more accurately the proper initial conditions. From test 1a results, the simulation start time should be approximately three hours prior to the beginning of the rainfall event, consequently the simulation start time of 8.30 was utilised.

The simulation results (fig. D.37) indicated that if the quality calculations have initialised prior to the start of the rainfall event, the ADWP, if input sensibly, does not have a significant effect on the peak flush. However, it is apparent that for even an ADWP difference of just three hours the model proved to be sensitive. This highlights the importance of a more suitable initialisation method than one based upon steady state calculations.

It was thought possible that improvements to the modelled outputs could be gained by increasing the ADWP due to its apparent sensitivity. However, increasing this parameter purely for the sake of modelled improvements constitutes a force fitting of the model. Consequently, the ADWP was increased from 26 hrs to 34hrs as this was considered the maximum possible ADWP which could be justified from the rainfall records. Figures D.38 and D.39 show marginal sensitivity to the modelled outputs however they are nowhere near the range which they need to be.

It can be concluded, from the tests, that the model outputs for the 31/5/95 event could not be improved by adjusting the simulations start time to allow either the qualitative calculations time to initialise, or by adjusting ADWP to account for the earlier simulation start time.

Note:-

The above tests were carried out with respect to a real rainfall event which occurred at 11:30am. As can be seen from the graphs, the modelled 'flush' proved insensitive to start time of the simulation and ADWP, provided the model had initialised (qualitatively) by the start of the rainfall event. This insensitivity however would have been because the flow rate between 08:30 and 11:30 was too high to cause deposition, or because the concentration of pollutants within the flow at these times (8:30, 9:30, 11:30) was insufficient to cause a substantial build up of pollutants. Consequently, it is possible that deposition would occur during periods of lower flow. The worst potential case being the time during the day which conveys a low flow rate, yet holds a significant concentration of pollutants i.e. afternoon. However, this was not investigated because the purpose of the tests was not to routinely test -QM, but to determine whether the inadequate modelled results could be improved. Consequently, such a sensitivity test, although interesting, would not significantly benefit the study.

12.5 Test 2 - Infiltration Modelling and Sensitivity of Storm Model

Outputs

As described in section 10.13 the sediment characteristics required to produce calibrated dry weather quality models whilst considering and neglecting infiltration were different. A test was therefore carried out to ascertain whether the two models would produce different pollutographs for any particular rainfall event. The sediment characteristics of the original -QM model (which did not consider infiltration) and the infiltration model are shown overleaf.

	Original Model	Infiltration Model
Diameter	0.15mm	0.5mm
Specific Gravity	1.3	1.035

Figure D.40 shows that the hydraulics of the system are modelled virtually the same with or without the consideration of infiltration. The corresponding storm pollutographs for the two models are shown in figures D.41 and D.42. It can be seen from these figures that the infiltration model produced a marginally better TSS pollutograph and a substantially better COD pollutograph. The modelled error with respect to the total TSS load was -82% when no account was given to infiltration. This error was improved by 2% when infiltration was considered. The modelled error with respect to the total COD load was -48% when no account was given to infiltration whereas, when infiltration was considered the modelled error was reduced to -33%. This therefore demonstrates the affects which the different sediment characteristics have on the storm simulation results.

12.6 Test 3 - Sediment Characteristics Sensitivity

12.61 Sediment Diameter Adjustment - Background, Discussion and Results

The principle adopted for the sediment diameter test was to reduce the diameter from the calibration value of 0.5mm to a minimum value which would arguably still provide a calibrated DWF model. **The infiltration model was used for these subsequent tests as the pollutographs for the infiltration model, as shown in test two, produced better results.** The storm event was then run and the outputs analysed to determine whether the pollutant profiles had improved.

12.62 Diameter Test Results

The minimum diameter which could be used for DWF modelling was 0.1mm. Values below this would not produce diurnally varying pollutograph profiles and thus did not represent a calibrated DWF model. The sensitivity of the QM model to the test characteristics (shown overleaf) can be seen in figures D.43, D.44, D.45 and D.46

Calibrated characteristics	0.5mm diameter / specific gravity 1.035
Test characteristics	0.1mm diameter / specific gravity 1.035

Figures D.43 and D.44 show that the model has remained accurate at the subcatchment sampling point, whilst figures D.45 and D.46 show the difference between the pollutographs at the system outfall. This therefore shows the effect of the sediment characteristic adjustment as the pollutants travel through the system. From these figures it can be seen that the outputs for both data sets (Dia. 0.5mm S.G. 1.035, dia. 0.1mm S.G. 1.035) represent a calibrated model, albeit the former calibrations allow the model to follow the observed data more closely.

12.63 Storm Sensitivity to Diameter Adjustment

The pollutographs shown in figures D.47 and D.48 show that the model was not sensitive to decreased diameter.

12.64 Specific Gravity Calibration

The diameter was returned to the calibration value of 0.5mm and the specific gravity reduced to a minimum value which would arguably provide a calibrated DWF model. The storm event was then run and the outputs analysed to determine whether the model would show greater sensitivity to specific gravity adjustments than to diameter adjustments.

The minimum specific gravity which could be used was 1.010. The sensitivity of the QM model to specific gravity is shown in figures D.49 and D.50.

From figures D.49 and D.50 it can be seen that with a specific gravity of 1.01 the model has still represented the sewer flow quality to a reasonable degree of accuracy at the subcatchment heads (sample subcatchment:- Bridgend). Figures D.51 and D.52 show the model is still representing suspended solids to a reasonable degree of accuracy at the system outfall, however it can be seen that TSS and COD were beginning to drift excessively (this was more apparent with COD than TSS, however specific gravity values lower than 1.01 produced

unstable results). This therefore highlighted that the model was no longer calibrated and thus the sensitivity test was extreme.

12.65 Storm Sensitivity to Specific Gravity Adjustments

Figures D.53 and D.54 show the models insensitivity to specific gravity calibration. The previous tests were concerned with decreasing the sediment characteristics (i.e. making the sediment 'lighter') to determine model sensitivity. It was also thought necessary to increase the particles weight to ascertain the models sensitivity. The way in which this test would work would be that theoretically more sediment would settle out of the flow during the ADWP, thus increasing the potential magnitude of any consequent flush. As the model proved marginally more sensitive to specific gravity than to diameter, the specific gravity of the sediment was increased. The largest specific gravity which could be used whilst retaining a calibrated DWF model was 1.1. Figures D.55 and D.56 show that the model is still calibrated at the system outfall.

Figures D.57 and D.58 highlight the models insensitivity to specific gravity adjustment under storm conditions..

Note:-

Sediment diameter and specific gravity were not adjusted simultaneously. This procedure was not adopted because if these parameters were adjusted simultaneously then, either specific gravity would have to increase with decreasing diameter, or diameter would have to increase with decreasing specific gravity in order to maintain a calibrated DWF model. The problem here being that the same 'effective' sediment characteristics as those for the calibrated model could be reproduced (i.e. smaller specific gravity but with a larger diameter), thus nullifying the test.

12.7 Sensitivity Test Conclusions - Summary

- To avoid qualitative modelling inaccuracies during storm modelling the simulation should be started approximately three hours before the beginning of the rainfall event. This duration provides the package with sufficient time to allow the quality calculations to reach a state of equilibrium.

Note: The duration may be related to the size and complexity of the drainage system. Consequently the three hour initialisation period, as defined above, may be site specific to the Perth drainage system.

- The adjustment of ADWP to account for the earlier simulation start time (3hrs prior to the rainfall event) does not have a significant effect on modelled outputs.
- The sediment characteristics required to produce a calibrated DWF model were different whilst accounting for, and whilst ignoring infiltration. The model which explicitly considered the effects of infiltration produced a marginally better storm quality model.
- Specific gravity and diameter adjustments could not improve the pollutant flush profiles which occurred during the large rainfall event (31/5/95).

12.8 BOD Sensitivity

- BOD proved significantly less sensitive to the TSS sensitivity tests compared to COD. The reason for this was that BOD had a much lower potency factor than COD (McGregeor, 1995), as demonstrated in table 7.3.

12.9 Infiltration Modelling Conclusions

As discussed in the sensitivity test conclusions, the infiltration model produced a more accurate storm quality model. Although the differences in pollutographs between the two models were not great, the sediment characteristics of the infiltration model appeared to respond better to increased flow rate, thus producing slightly better results. Consequently, it is believed that the full benefit of infiltration modelling will not be seen using this version of the -QM software as it does not consider deposited pipe sediment erosion processes. If deposited pipe sediment were accounted for then its erosion would mean more of the 'more

responsive' sediment would be released into the flow. This could significantly improve the TSS modelled profile.

12.10 Hydroworks -QM - Overall Conclusions

Dry Weather Quality

TSS, BOD and COD, under dry weather conditions, were modelled acceptably. The modelling of ammonia however proved more problematic as modelled ammonia at the system outfall was over predicted. The problem could not be remedied via calibration and consequently the problem was attributed to a dilution effect caused by infiltration. Consequently, dry weather flows and pollutants were re-modelled with explicit consideration given to the infiltration flows. TSS and COD profiles were marginally improved using the new infiltration model, however ammonia results at the system outfall remained high. The problem was therefore attributed to a software 'glitch'.

Storm Quality

During the low intensity rainfall events, 24/8/95 and 29/8/95, which were less than 2mm/hr, the general trends of the pollutant profiles for TSS, BOD and COD were represented acceptably, although correlations between modelled and observed data sets were poor. The model outputs for the large rainfall event 31/5/95 (peak intensity >40mm/hr), were very poor for TSS in the Bridgend subcatchment, although BOD and COD were modelled much better. The reason for this however, is not clear, although the problem is believed to be the lack of model accountability for the erosion of deposited pipe sediment. The modelling of ammonia during storms was problematic, as expected. This was attributed to the poor modelling of ammonia under DWF conditions. As the DWF ammonia results could not be improved via calibration, the substandard model storm outputs required to be accepted. Sensitivity testing was carried out to ascertain whether the modelled outputs, for the high intensity event, could be improved by sediment characteristic calibration, however this proved wholly unsuccessful.

12.11 The Inadequate Modelling of TSS

The possible reasons for TSS being under predicted, with COD and BOD being modelled more acceptably are; granular deposited pipe sediment erosion, or that the surface washoff quality model is inadequate, or a combination of both.

It is not thought that the poor TSS modelling is a consequence of an inadequate representation of surface washoff quality as no gross under prediction of pollutants occurred in the other sub-catchments which do not suffer from sedimentation. If the problem lay with the gully model then modelling inaccuracies would be most apparent in the subcatchments which were not prone to sedimentation. However this proved not to be the case; pollutant modelling was reasonably accurate in Craigie (which is not prone to sedimentation) and inaccurate in Bridgend (which *is* affected by sedimentation). In addition all determinands, TSS, BOD and COD were modelled reasonably well in Craigie suggesting that the potency factors associated with gully input/outputs are acceptable. This therefore discounts, partially if not fully, the hypothesis of a poorly calibrated gully pot model and substantiates the hypothesis that the additional sediment, which was not modelled in the Bridgend subcatchment, came from the erosion of deposited pipe sediment. Consequently, the most likely explanation is that erosion of deposited sediment did occur, but that the eroded sediment was of a very granular nature (otherwise -QM would also have under predicted BOD and COD). This, however does not necessarily appear to be the case from collected sediment data (Tables 8.1, 8.2 and 12.1). However, if the temporal variations in the pollutant concentration are taken into consideration, this hypothesis could make sense. Substantial temporal variations are known to occur in sediment concentrations (**Jack et al, 1995**), therefore it is possible that the concentration of BOD and COD on the day of the event were very low in relation to the concentrations on the day of the data collection exercise. This is a strong possibility as the ADWP for the 31/5/95 event was only twenty six hours with the preceding event having a peak intensity of 14mm/hr. Such an event could have eroded the previously deposited organic matter, leaving mainly granular sediment.

12.12 Summary/ Conclusions

During the low intensity rainfall events, weak correlation existed between modelled and observed data sets. During the more extreme rainfall event, where it is possible that deposited pipe sediment has been eroded, -QM grossly underestimated the TSS flush. BOD and COD are apparently modelled substantially better, suggesting that the sediment deposits in the Bridgend subcatchment, on the day of the event, were low. It was therefore concluded that although -QM, in its present form, is a limited sediment transport model, BOD and COD can be modelled reasonably well, provided the pollutants within the deposited sediment remain low. As this is unlikely to be the case, -QM can be expected to substantially under predict pollutant concentrations for most major events throughout the year. Four options were therefore left with respect to the modelling of sewer flow quality:-

1. Work with Wallingford software in an attempt to develop a suitable deposited sediment bed model;
2. Discard Hydroworks-QM and convert to another sewer flow quality package - MOUSETRAP;
3. Develop a non deterministic sewer flow quality model
4. Utilise a simplistic modelling approach which is no less accurate than either -QM or MOUSETRAP.

Each possibility is discussed in detail below.

12.121 Development of a Suitable Deposited Sediment Bed Model

As the processes of sediment erosion and pollutant release from deposited pipe sediment, during storm conditions, are believed to contribute a very substantial component of the discharged pollutant load to the receiving watercourse the processes must be well represented within the package if the model is to have any integrity. Unfortunately, significant limitations exist, with regard even to the most fundamental areas of these processes e.g. sediment transport. Nevertheless, even if it can be assumed that one day the gaps in knowledge will be rectified and a superior equation/approach proposed, the difficulties of dealing with the

complex variability of the pollutants attached to the sediments being transported still remains.

Given the high degree of variability in pollutant concentration, a deterministic sewer flow quality model ideally requires the definition of a sediment characteristic file which will represent the average characteristics of the sediments for each pipe in which sediment exists. This is more suited to the deterministic sewer flow quality model MOUSETRAP (Garsdal et al, 1995) due to the inherent greater flexibility. Nevertheless, sewer sediment deposits are prone, not only to spatial variation (tables 8.1 and 8.2), but temporal variation (table 12.1) as well (Ashley, 1993). This indicates that *all* sediment data should be obtained from samples collected from the catchment on the same day, *and* at the same time. The importance of this is for setting up the correct initial conditions in the model. Unfortunately this would be highly impractical, if not impossible to achieve.

Table 12.1 Temporal & Spatial Variability Of Pollutants Attached To Sediments (Within Single Pipe Length ~ 48m)

Sampling Point	1/12/94 BOD(mg/l)	16/12/94 BOD(mg/l)	13/1/95 BOD(mg/l)	27/1/95 BOD(mg/l)
Site 1	7150	14300	5130	6880
Site 2	1650	1500	5500	5320
Site 3	3850	4500	4300	4500
Site 4	7430	7510	4220	2750

Data collected from Dundee Int. Sewer
Source:- (Hutchison, 1995)

Although it is unlikely that the awaited -QM bed load model will contain the flexibility to define sediment characteristics for each particular pipe affected by sedimentation, it is Wallingford Software's intention to address the problem of 'what pollutant concentrations are held within the new bed, which rapidly re-establish after an event which has caused erosion (Wotherspoon, 1995), - and how do these concentrations change with time?'. This is a very important area, but unfortunately one in which the answers at present simply do not exist. A great deal of research is therefore required and may take at least ten years before suitable answers begin to be found. Consequently, there would be no benefit, as

far as this research project is concerned, in working with Wallingford software to develop such a model.

12.122 Utilisation of MOUSETRAP

MOUSETRAP is similar to the Hydroworks - QM package. Both models are based on full solution hydraulic models and thus adopt a deterministic approach to the modelling of sewer flow quality. MOUSETRAP however, is a much more versatile package as a range of transport equations are available, allowing localised control of the sediment and pollutant characteristics. Furthermore, MOUSETRAP explicitly accounts for degradation/water quality process which occur as pollutants travel through the drainage system, whereas -QM simply routes the pollutants as they travel throughout the system. The MOUSETRAP model however, like Hydroworks -QM, was also under evaluation, and as the limitations of knowledge which frustrate accurate sewer flow quality modelling also apply to MOUSETRAP, there was no guarantee that a MOUSETRAP model of the Perth system would be any more accurate than the -QM model. The additional time required to convert to a MOUSETRAP model would only be justified if success could be guaranteed. Consequently conversion to MOUSETRAP required to be ruled out.

12.123 Non Deterministic Modelling

The nature of the problems facing quality modelling suggests that a stochastic, empirical or conceptual model could be utilised in preference to the deterministic platform. Unfortunately, stochastic or empirical models require very large data sets to justify model verification and due to the inherent difficulties associated with storm data collection this was not the most practical option (**Jack and Petrie, 1995**). As correlations are based on existing conditions, they are not suited to engineers requiring to analyse potential system performance via changes to the original system (**Phan et al, 1994**). Conceptual and deterministic models, however, allow 'what-if' scenarios to be tried and tested relatively easily. As understanding of the in-sewer processes are limited, the conceptual model is 'theoretically' a more practical tool. SIMPOL is the UPM simplified/conceptual

modelling package, which takes the form of a spreadsheet model. The initial motivation for development was the prolonged computational times required to run rainfall sequences and/or continuous simulation using deterministic models. The essence of SIMPOL is to represent the various elements of a sewerage system via surface, sewer, CSO and storm 'tanks'. Each tank provides a very simplistic representation of the aforementioned areas with the calculations generally being carried out using transfer functions. The model can be calibrated against either, a detailed deterministic model, or directly from collected data. Due to the errors and uncertainties associated with the deterministic modelling, it is believed it would be more appropriate to construct a SIMPOL model using collected data, thus eliminating any potential iterative modelling errors.

It was decided, however, not to construct a SIMPOL quality model of the Perth sewerage system as the storm data collection period, which lasted, in total, a period of 15 months produced only 'patchy' data which were not believed sufficient for the calibration of a conceptual quality model.

12.124 Simplistic Modelling Approach

Due to the aforementioned modelling problems, it was decided that **the most suitable method of representing sewer flow quality would be achieved via the utilisation of hypothetical qualitative data**. Although this is not the recommended approach for complex modelling it was deemed to be the most appropriate method given the circumstances. The derivation of the hypothetical data is detailed in chapter fifteen.

Chapter Thirteen

Waste Water Treatment Plant Modelling

13.1 Introduction

The first Perth WTP model was constructed using a prototype version of WRC's STOAT software for use in the initial research programme (**Petrie, 1997**). This model was handed over by NoSWA to the University of Abertay Dundee in a near completed state with only storm tank calibrations requiring to be carried out. This chapter reviews the calibration of the STOAT model and considers its potential for use within this research project.

13.2 Sleepless Inch Waste Water Treatment Plant

Each of the unit processes as described in chapter six were modelled as a single unit incorporating the total volume and surface area of the actual units. This is justifiable because simplifying hydraulic assumptions are made, and as a result the larger unit is equivalent to the several smaller ones (**Bryan, 1993**).

13.3 Data Collection

Dry weather flow and quality data for the model were collected on the 27th and 28th April, 1993. These data were used to represent a typical day of dry weather inputs for the model. The data were therefore used as a profile to provide time series of several days length, allowing long runs to be performed. Initial conditions were determined for each of the unit processes by copying from a model of a similar works. This allowed the individual model processes to start from an equilibrium whilst avoiding the long convergence times associated with guessing values for an initial state or starting with an 'empty' system (**Bryan, 1993**).

13.4 Model Calibration

The primary tanks and activated sludge plant were calibrated concurrently since the surplus sludge from the activated sludge plant is co-settled in the primary tanks. The primary tanks were initially modelled with one continuously stirred tank reactor (CSTR), however the number of CSTR's required to be increased until the timing and amplitude of the ammonia predictions matched the observed data (**Bryan, 1993**). Four CSTR's were required. The suspended solids in the effluent are predicted using the formula as shown in eqn 13.1 below:-

$$V = K_b C_s^h \quad \text{eqn. 13.1}$$

where V = settling velocity (m/h)

C_s = settleable solids concentration (kg/m³)

K_b & h = coefficients

In order to calibrate the TSS, the fraction of the solids which could settle required to be increased to 90% and the coefficient K increased to 2.5. The dry weather plots showing these calibrations are shown in figures E.1 to E.6. The average modelled error for TSS concentration in the primary tanks was calculated as +2% (fig. E.1). The maximum error over the twenty four hour period occurred at hour 6 and was +53%. With respect to the peak error, the observed peak concentration occurred at hour 12 and was 145mg/l. The modelled peak occurred hour 8 and was 147mg/l. This produced an error of +1.4%.

The average modelled error with respect to BOD was -4% (fig. E.2). The maximum error was calculated as +38%. This error also occurred at hour 6. The peak sampled concentration occurred at hour 12 and was 170mg/l. The peak modelled concentration occurred two hours earlier and was 180mg/l. Consequently, the error with respect to the peak was +5.88%. The average modelled error for ammonia (fig. E.3) was -4%, however, the maximum error was -12%. This error occurred at hour 20. The peak sampled concentration occurred at hour 8 and was 20mg/l. The peak modelled concentration occurred at hour 10 and was also 20mg/l. This meant that there was no peak error.

Dissolved oxygen (DO) control was used for the activated sludge plant. The DO set points for each pocket were set to 1mg/l, 1.5mg/l and 4mg/l respectively. This proved reasonable and correlated with observations made at Sleepless Inch for the three respective stages. The maximum value for oxygen transfer (K_{la}) for each pocket was set to 12. The wastage rate was originally set at a constant rate of 3.2l/s, estimated from observed data, however this resulted in an under-prediction of the mixed liquor suspended solids (MLSS) and an over-prediction of the waste activated sludge (WAS). Wastage control was therefore set to set point MLSS control which provided more suitable effluent and WAS concentrations. It was important to ensure an accurate representation was made of MLSS as this is the fundamental parameter responsible for the degradation of the wastewater. The reason for this is that the volatile fraction of the MLSS is taken as an approximation of the active biological mass (**Metcalf and Eddy, 1991**). The maximum possible settling velocity (V_o) and the exponential constant for hindered settling (k) for the final settlement tanks were estimated using Specific Stirred Volume Index (SSVI) data. The value for V_o was 1.52 m/h, compared to the default of 5.625m/h. This proved to be too low and therefore V_o was increased to 2.5m/h. The value of k was calculated to be 0.00051 using equation 13.2.

$$k = (0.000269 + 0.00122SSVI_{3.5}) \quad \text{eqn 13.2}$$

where

SSVI_{3.5} = Specific Stirred Volume Index at a concentration of 3.5kg/m³

The exponential constant for settling at low solids concentration (p) was increased to 0.03. This parameter cannot be measured and is therefore considered to be a calibration factor (**Bryan, 1993**). These calibrations resulted in a very good match for final effluent ammonia under dry weather conditions (Fig. E.6). However, this was expected as the plant does not nitrify. The average modelled error with respect to the ammonia concentrations was calculated as +3%. A maximum error of +14% occurred at hour zero. The peak sampled concentration occurred at hour 22 and was 17.8mg/l. As it was not believed that a time lag was

present, the modelled error with respect to the peak was calculated as -8.9% . This figure was based on the modelled concentration of 16.2mg/l at hour 22. The average error with respect to the modelled TSS concentrations in the final effluent was $+38\%$ (fig. E.4). The maximum error occurred at hour 10 and was $+100\%$. The peak TSS concentration was sampled at hour 2 and was 19mg/l . The modelled peak occurred also occurred at hour 2 and was 20mg/l . This produced an error with respect to the peak of $+5.3\%$. The maximum error with respect to BOD was $+178\%$ at hour 12. This is shown in figure E.5. The average error was calculated as $+86\%$. The peak BOD concentration was sampled at hour 18 and was 10.2mg/l . The modelled peak was however 21mg/l at hour 14. This resulted in a peak error of $+105.9\%$.

13.5 Storm Modelling

Storm flows tests the applicability of the calibrated dry weather flow model to more extreme circumstances. Only one storm event, sufficient to cause a storm overflow was recorded at the front end of the data collection exercise (19/4/93). Although calibration of the activated sludge process had not been completed to an acceptable level, runs were performed in order to give some verification results. These were performed using a "five day" storm weather time series (18/4/93 to 23/4/93).

13.51 Primary Tanks (Storms)

The average modelled error with respect to TSS concentration was calculated as -6% , however the maximum error was $+54\%$ at hour 25. This is demonstrated in figure E.7. The peak TSS concentration was sampled at hour 80 and was 160mg/l . The modelled peak occurred at the same time-step and was 178mg/l . This produced a peak error of $+11.25\%$.

The average modelled BOD error was $+7\%$, although the maximum error was $+212\%$ at hour 105 (figure E.8). This error coincided with high influent SS and BOD in the crude sewage, possibly from the first flush of the subsequent storm. It is possible that a higher than normal proportion of the BOD was associated with solids leading to the over-prediction. The peak BOD concentration was

sampled at hour 80 and 135mg/l. The peak error was therefore +9.6%. This was based on the modelled concentration of 148mg/l at the same time-step. However, it can be seen that the maximum error at hour 105 was greater than the peak error. This highlights problems with the representation of BOD in the activated sludge process. The average modelled error with respect to ammonia was +7%. The maximum error was +33% at hour zero (fig. E.9). Timing of the predicted and measured data was reasonable as the modelled and sampled peaks occurred at the same time step (hour 100). The sampled peak was 16.4mg/l and the modelled peak was 15.8mg/l. This produced an error of -3.6%.

13.52 Activated Sludge (Storms)

The predicted and measured results from the ASP under storm conditions are shown in figures 13.1 to 13.3

Fig. 13.1 TSS in Activated Sludge Effluent (STOAT - Storm)

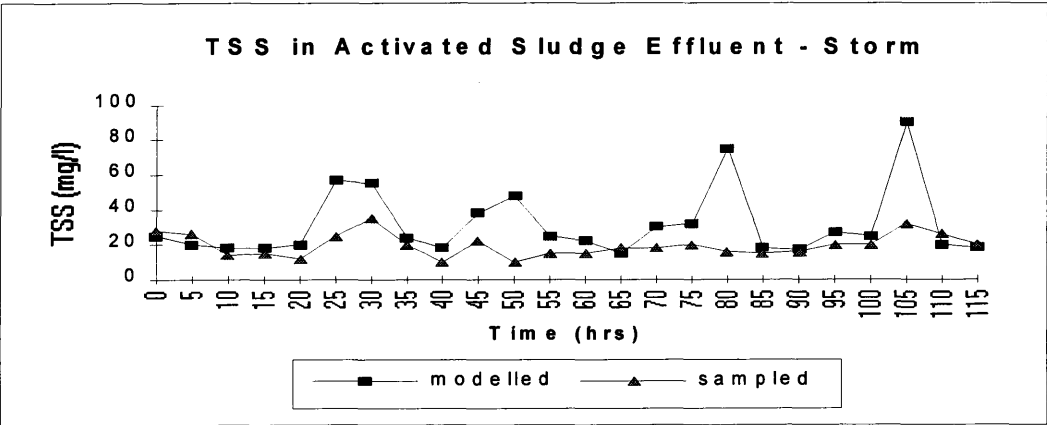


Fig. 13.2 BOD in Activated Sludge Effluent (STOAT - Storm)

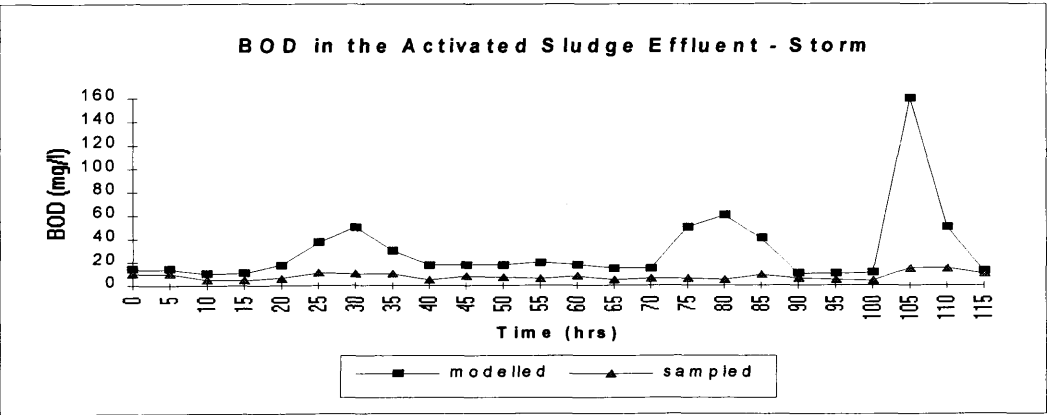
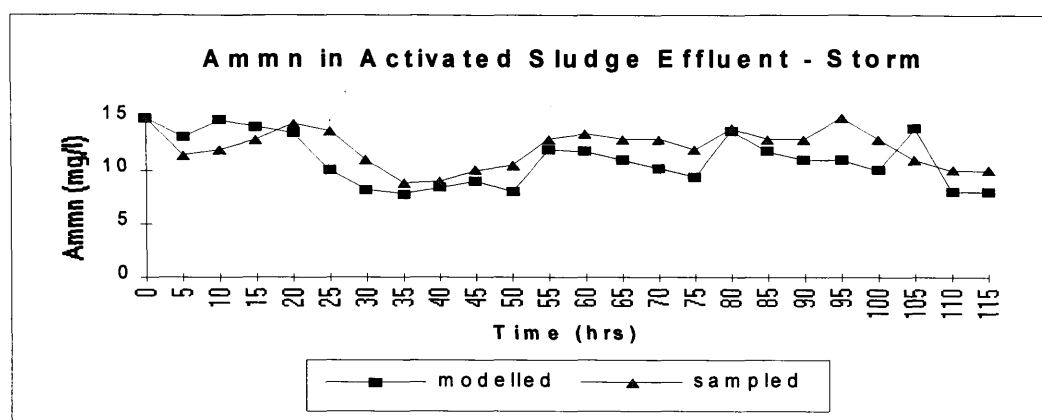


Fig. 13.3 Ammonia in Activated Sludge Effluent (STOAT - Storm)



The average modelled error for ammonia was -10% with a maximum error of -28% at hour 25 (Fig. 13.3). The sampled peak occurred at hour 95 and was 15mg/l. The modelled profile under-predicted this peak as a modelled concentration of 11mg/l was obtained. This produced an error of -26.6%. The average modelled error for TSS and BOD was +71% and +204%, respectively. The respective maximum errors for the two determinands were +380% and +966.6% (figures 13.1 and 13.2). The peak sampled TSS concentration occurred at hour 30 and was 35mg/l. The model predicted a concentration of 55mg/l producing an error of +57.1%. The peak sampled BOD concentration occurred at hour 105 and was 15mg/l. The modelled peak occurred at the same time-step but was 160mg/l. This produced an error of +966.6%, which coincided with the maximum BOD error. It was noted that the waste activated sludge flows were reduced to zero for several days. This was due to the high MLSS set point, which was set in relation to the calibration day when MLSS levels were 25 to 50% higher than the first 5 days of monitoring. The high MLSS set point was probably a significant cause of the mismatch in predicted and measured data. This clearly highlights the problem of a non-scientifically controlled system.

13.6 STOAT Model Summary and Conclusions

Primary Tanks

Table 13.1 highlights the modelled errors observed in the primary tanks effluent under dry and storm conditions.

Table 13.1 Average and Peak Modelled Errors in Primary Tank

Primary Tanks	TSS	BOD	Ammn
Average Error (DWF)	+2%	-4%	-4%
Average Error (Storm)	+6%	+7%	+7%
Peak Error (DWF)	+1.4%	+5.88%	0%
Peak Error (Storm)	11.25%	9.6%	-3.6%

It can be seen from table 13.1 that the average modelled errors for all determinands are less than +/-10% for dry weather and storm conditions. It can also be seen that the peaks have also been modelled well as the largest error was only 11%. This indicates that a good representation of the primary settlement tank has been made.

Table 13.2 highlights the modelled errors observed in the activated sludge effluent under dry and storm conditions.

Table 13.2 Average and Peak Modelled Errors in Activated Sludge Effluent

Activated Sludge Effluent	TSS	BOD	Ammn
Average Error (DWF)	+38%	+86%	+3%
Average Error (Storm)	+71%	+204%	-10%
Peak Error (DWF)	+5.3%	+105.9%	-8.9%
Peak Error (Storm)	+57.1%	+966.6%	-26.6%

It can be seen from a comparison of table 13.1 and 13.2 that the average TSS and BOD errors in the activated sludge effluent were significantly greater than those obtained in the primary tank effluent. This was true even for dry weather flows. It can also be seen from table 13.2 that problems occurred predicting the peaks, principally with respect to TSS and BOD under storm conditions. The TSS over-predictions coincided with peaks in settled sewage. This problem was believed due to the critical MLSS parameter being too high. Some of the BOD over-

prediction was associated with this problem however, the model still significantly over-predicted BOD at periods of high flow. This is a recognised weakness of the STOAT model and is due to the poor applicability of the activated sludge model (ASAL1) for BOD at low retention times (**Bryan, 1993**). The reason for this over-prediction is due to an assumption that particulate material is hydrolysed immediately (**Dudley, 1996**). This resulted in a greater portion of readily available substrate being available within the reactor than that which existed in reality (actual hydrolysis processes are slow). Consequently, during periods of high hydraulic loading, when the retention time within the tank was not sufficient to treat the additional soluble BOD, over-predictions occurred.

Table 13.2 shows that Ammonia in the activated sludge effluent was modelled to a significantly higher level of accuracy than TSS and BOD. Although, this was expected as the plant did not nitrify.

It was concluded from the above analysis that the primary tanks within STOAT provided an adequate representation of the processes which occur within this unit process. However, the representation of the activated sludge was significantly less accurate. This was believed to be due two principal problems. The first was that the model over-predicted MLSS concentrations within the reactor. This partly caused the TSS and BOD over-predictions in the effluent. This was however, not a limitation of STOAT, but a simple inherit modelling problem associated with modelling a non-scientifically controlled WTP. The second problem was associated with the hydrolysis assumptions made within the prototype STOAT software as discussed above. This assumption/limitation was partly responsible for the over-prediction of BOD in the activated sludge effluent under storm conditions. Due to this modelling limitation it was decided to convert the prototype STOAT model to the more comprehensive treatment plant model GPS-X. This was carried out in order to ascertain whether GPS-X could provide more accurate results. A comprehensive evaluation of GPS-X is provided in the following chapters.

Chapter Fourteen

WTP Modelling - GPS-X (Theory)

14.1 Introduction

GPS-X is a multipurpose modelling system which allows the performance of WTP's to be simulated, controlled and optimised. The package contains a variety of different steady state and dynamic models, including the widely used IAWQ activated sludge model number one (Henze et al, 1986). This chapter details the theory behind GPS-X and discusses the calibration of the Perth GPS-X model. The objective of this work was to ascertain whether GPS-X could provide better results than the prototype version of STOAT and therefore be of greater use to this research project. Consequently, the results from both GPS-X and STOAT are compared and contrasted in this chapter.

14.2 Data Requirements

The basic data requirements for GPS-X are similar to that of STOAT, although certain differences are evident. User defined kinetic and stoichiometric data can be defined within GPS-X thus making GPS-X somewhat more flexible than the prototype STOAT model used in this study (although STOAT has subsequently been redeveloped).

14.3 GPS-X Libraries

GPS-X libraries define the state variables (variables continuously integrated over time) which are to be used. Within GPS-X four libraries exist as listed below:-

- Carbon Nitrogen library 1(CN1)
- Carbon Nitrogen Phosphorous library (CNP)

- Carbon Nitrogen library 2 (CN2)
- Industrial Process library (IP)

The CN library is the simplest and is therefore the default library. Within the CN library, 12 state variables are modelled. The state variables and their corresponding symbols are shown in table 14.1

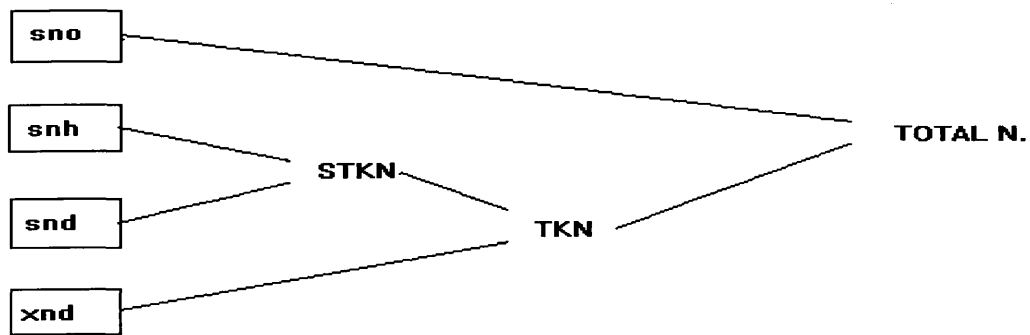
Table 14.1 CN Library State Variables

No.	State Variables	Symbol
1	Readily Biodegradable (Soluble) substrate	ss
2	Slowly biodegradable (particulate) substrate	xs
3	Particulate organic inerts	xi
4	Soluble organic inerts	xs
5	Active heterotrophic biomass	xbh
6	Active autotrophic biomass	xba
7	Cell residue from decay	xu
8	Dissolved oxygen	so
9	Nitrate and nitrite	sno
10	Ammonia and Ammonium	snh
11	Soluble biodegradeable organic nitrogen	snd
12	Particulate biodegradable organic nitrogen	xnd

14.31 Calculations

The soluble TKN (STKN) is the sum of the ammonia (snh) and the soluble organic nitrogen (snd) and the TKN is the sum of the soluble TKN and the particulate organic nitrogen (xnd). The total nitrogen is the sum of the TKN and nitrate/nitrite nitrogen (sno). Inorganic nitrogen is not considered. This is shown diagrammatically in figure 14.1

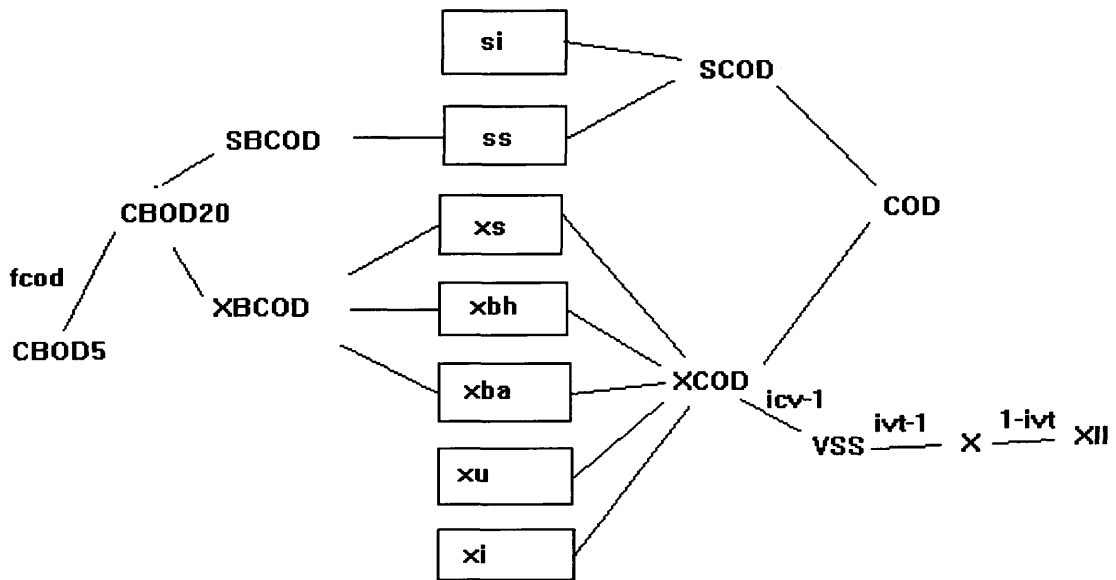
Fig. 14.1 Total Nitrogen Calculation



With respect to the COD and suspended solids composite variables (fig. 14.2), the relationship is more complex, involving stoichiometric fractions as well as state variables. Soluble COD (SCOD) is the sum of the soluble inerts (si) and soluble substrate (ss), while the particulate COD (XCOD) is the sum of the slowly biodegradable particulate substrate (xs), active heterotrophic biomass, active biomass (xba), cell residue from decay (xu) and particulate inert organics (xi). The soluble and particulate COD sum to give the total COD.

Suspended solids are calculated from the particulate COD (XCOD) by dividing by the XCOD:VSS ratio (icv) which changes the units of the XCOD to mg VSS/l. This results in the composite variable for volatile suspended solids (VSS). To calculate the suspended solids composite variable (X), the VSS is divided by the VSS:TSS ratio (ivt). The inert inorganic particulate (xii) is calculated from (1-ivt) multiplied by the suspended solids composite variable (X). Carbonaceous BOD (CBOD) is also calculated from state variables. The biodegradable state variables (the state variables that exert CBOD) are added to provide both the particulate and soluble biodegradable COD value. The sum of these components, which is the total biodegradable COD measurement, is assumed to be equivalent to CBOD₂₀ or the ultimate CBOD. In order to determine the amount of CBOD₅ a stiochiometric fraction, fbod, which is the ratio of CBOD₅:CBOD₂₀ multiplies the calculated CBOD₂₀.

Fig. 14.2 COD Calculation



The CNP library allows the modelling of an additional five state variables, the CN2 library, an additional seven, and the IP library an additional ten (**GPS-X Tech. Ref, 1994**). However, the twelve state variables modelled within the CN library were deemed sufficient to allow representative modelling of the basic water quality parameters BOD, COD, TSS and Ammonia (**GPS-X User's Guide, 1994**). This library was therefore used for the construction of the Perth WTP model.

14.4 Unit Process Models

14.41 Clarifier Models

A variety of one or two dimensional settler models can be chosen. All are based on standard solid flux theory in which the movement of solids in a settling basin are controlled by gravity and underflow rate. The models are applicable to both steady state and dynamic simulations. The one dimensional settler model is divided, typically into ten layers of constant thickness. A mass balance is carried out around each layer in the settler model, subject to the following assumptions

- Incoming solids are distributed instantaneously and uniformly across the entire cross-sectional area of the clarifier feed layer.

- Only vertical flow is considered.

The model uses traditional solids flux analysis with a limit on the downward solids flux to the level that can be handled in the layer below. The settling velocity model is shown below:-

$$V_{sj} = V_{so} e^{-rhin x_j^0} - V_{so} e^{-rflo x_j^0} \quad \text{eqn. 14.1}$$

where

V_{sj} is the settling velocity in layer j (m/d),

V_{so} is the maximum Vesilind settling velocity (m/d)

$rhin$ is the hindered settling zone parameter (m^3/g)

$rflo$ is the flocculant zone settling parameter (m^3/d)

$$x_j^0 = x_j - x_{min} \quad \text{eqn. 14.2}$$

where

x_j is the suspended solids concentration in layer j (mg/l).

x_{min} is the minimum attainable suspended solids concentration (mg/l).

The Vesilind settling parameter is used where hindered settling occurs.

14.42 Modelling Flow Distribution in One Dimensional Settling Tanks

During normal flow conditions, the load enters the settler via the feed point. Normal loading is represented, mathematically, when the influent flow to surface area ratio is less than the *quiescent zone maximum velocity*. However, as the influent flow increases, the load to the settler begins to be distributed to the layers below the feed point. When the upflow in the settler surpasses the maximum upflow velocity, the load completely enters the bottom of the settler. When the upflow velocity is between the quiescent and maximum upflow velocity, the loading position is determined using linear interpolation between the two limits. This procedure attempts to represent the feed distribution to the settler with respect to different volumetric loading conditions i.e. during periods of high

loading the momentum of the incoming flow effectively changes the feed point in the settler from normal loading conditions.

There are four settlement models which can be chosen; Mantis 1D, General 1D, Simple 1D and No React 1D. The difference between the Mantis 1D and the General 1D models is that different biological models are used. The general 1D model, uses a General biological model and the Mantis 1D uses the Mantis biological model. The reactive calculations are based on the influent sewage quality characteristics (stoichiometry and dissolved oxygen).

The differences between the two non reactive type models, (No React 1D and Simple 1D) is that the No React 1D will account for the effects of dilution on the influent stream while the simple 1D model simply maps the composition from the influent to the effluent. A difference between these two models will only become apparent if the influent loading has a sharp change in stoichiometry (e.g. if there is a sudden rise in particulate inert material).

14.43 Two Dimensional Settler Model

A two dimensional rectangular clarifier model is currently available in GPS-X. The model combines the solids flux curve of the one dimensional settler model with a series of static or discrete flow fields. This is done using a computational fluid dynamics package (r2dclar). The advantage of the 2D model over the 1D model occurs during large transient flow conditions as a better prediction of peak suspended solids is gained. The 2D model however was not used for the Perth study as the 2D clarifier model available within GPS-X is for rectangular clarifiers, whereas the settlement tanks at the Sleepless Inch WTP (Perth) are circular.

14.44 Biological Models

A variety of biological models are available within GPS-X:-

- Conceptual

- General
- IAWQ
- MANTIS

Full details of the differences between each model can be found within the **GPS-X Technical Reference Manual, (1994)**. As the purpose of the research project, was not to evaluate and compare each different wastewater treatment plant model, but to use one model which could reliably represent the processes occurring at a WTP during dry and wet weather conditions the IAWQ Activated Sludge Model No.1 was chosen. This model is both well established and internationally recognised.

14.5 Model Construction - DWF

14.51 Primary Settlement Tank Modelling

The No React 1D model was found to be the most suitable for the primary sedimentation processes. The physical data were copied from the original STOAT model and input into GPS-X. The only additional data required was the feed point of the influent sewage relative to the top of the tank. These data were taken from on site measurements. No problems were encountered during DWF calibration for this unit process.

Figure E.10 shows the modelling of TSS in the primary tank effluent. The average modelled error was -11% and the maximum error was -38%. This error occurred at hour 16. The peak sampled concentration occurred at hour 12 and was 145mg/l. The peak modelled concentration occurred two hours earlier and was 146mg/l. This produced a peak error of less than 1%. The modelled BOD profile for the primary tank effluent is shown in figure E.11. The average modelled error for the BOD predictions was -1%. The maximum error occurred at hour 6 and was +45%. The peak sampled concentration occurred at hour 12 and was 170mg/l. The modelled peak occurred at hour 10 and was 168mg/l thus producing a peak error of -1.2%.

Fig.E.12 shows the modelling of ammonia. The average modelled error was –4% and the maximum error was –12% at hour 20. The peak sampled concentration occurred at hour 8 and was 20mg/l. The modelled peak occurred at hour 10 and was 20 mg/l. Consequently, there was no peak error with respect to ammonia.

14.52 **Activated Sludge and Final Clarifier Modelling**

Calibration of the activated sludge tank was carried out concurrently with the final settlement tank modelling. This procedure was adopted because the two unit processes operate as one complete unit. It was thus pragmatic to model the two unit processes simultaneously. The physical characteristics of the aerator and final settlement tanks were again copied from the STOAT model along with the initial state conditions.

A comparison of the settling characteristics which were required to calibrate both the STOAT and GPS-X final settlement tanks are shown in table 14.2.

Table 14.2 Settling Tank Calibration Parameters

Model/Parameter	STOAT	GPS-X
Max. Settling Velocity	2.52m/hr	2.48m/hr
Hindered settling parameter	0.000513	0.000513
Flocculent settling parameter	0.03	0.03

The default stoichiometric data proposed within GPS-X (e.g. ratios of particulate COD to volatile suspended solid ratios, volatile suspended solids to total suspended solids etc.) were utilised, along with the proposed default kinetic parameters (e.g. maximum specific growth rates and half saturation coefficients etc.). No problems occurred during DWF calibrations. The corresponding plots can be seen in figures E.13 to E.15.

The settling characteristics were copied from the STOAT model along with wastage and return activated sludge pumping rates. Figure E.13 shows the

modelling of TSS in the activated sludge effluent. It can be seen that the maximum modelled error was +80%. This error occurred at hour 24. The average modelled error was calculated as +20%. The peak sampled concentration occurred at hour 2 and was 19mg/l. The modelled peak occurred at hour 4 and was 16.3mg/l. This resulted in a peak error of -14.2%.

Figure E.14 shows the accuracy of the GPS-X BOD model under dry weather conditions. The maximum error was +62% at hour 4. The average modelled error was +17%. The peak sampled concentration occurred at hour 18 and was 10.2 mg/l. This peak was under predicted by the model and an error of -49.2% was obtained. The error was based on the modelled concentration of 5.18mg/l at the same time-step.

Figure E.15 shows the high accuracy of the ammonia results (which was expected as the plant does not nitrify). The average modelled error was +3%. The maximum error occurred at hour 0 and was +14%. The peak sampled concentration occurred at hour 22 and was 17.8 mg/l. The modelled peak occurred at hour 16 and was also 17.8mg/l. Consequently, there was no peak error with respect to ammonia.

14.6 Model Verification - Storms

The same five days of storm data (18/4/93-23/4/93) used to calibrate the STOAT model were run through the GPS-X model. Observed and predicted data sets are shown in figures E.16 to E.18 (primary tanks) and figures 14.3 to 14.5 (activated sludge effluent).

Fig. E.16 shows that in general a good representation of TSS is made in the primary tank effluent. The average modelled error was -4.5%. The maximum error occurred at hour 0 and was +71%. A similar, albeit, lesser error occurred at hour 95 and was -60%. It can be seen from figure E.19 that the modelled peaks are very close to the observed peaks with the exception of the period between 28-43 hrs. The largest error within this period was +59%. This problem could not be

remedied without reducing the accuracy of the overall predictions and consequently the error required to be accepted. The peak sampled concentration occurred at hour 80 and was 160mg/l. The model predicted a concentration of 174mg/l producing an error of +8.75%.

Figure E.17 shows the BOD in the primary tank effluent. The maximum error occurred at hour 105 and was +147%. The average error was calculated as +4%. The sampled peak occurred at hour 80 and was 135mg/l. The model over-predicted this peak by +15.5% (based on the predicted concentration of 156mg/l at the same time-step).

Figure E.18 shows the modelling of ammonia in the effluent from the primary sedimentation tank. The average error was calculated as -7% and the maximum error was +33%. This error occurred at hour 0. The sampled peak occurred at hour 100 and was 16.4 mg/l. The model under-predicted this peak by -3.6% based on the modelled concentration of 15.8mg/l at hour 100.

Figure 14.3 shows the comparison between the modelled and observed TSS in the final clarifier effluent. It can be seen that the modelled data are, at certain times, over-predicted. These peaks were also noted with the STOAT model and were believed to be a consequence of the problems associated with representing the MLSS control regime utilised at the WTP. The maximum error was observed to be +267% at hour 50. An average modelled error of + 32% was obtained. The peak sampled concentration was 35mg/l at hour 30. GPS-X over-predicted this peak by 51.4%. The over-prediction was calculated based on the model concentration of 53mg/l at hour 25.

Figure 14.4 shows the modelling of BOD data in the activated sludge effluent. It can be seen from figure 14.2, that BOD is principally over-predicted at hours 25-30 and hours 100-115. The maximum errors within these time steps are +144% and +114.2%, respectively. These periods correspond to the time steps where the STOAT model also over-predicted BOD concentrations. This is related to the MLSS modelling problem. The average modelled error was +69%. The peak sampled concentration occurred at hour 105 and was 15mg/l. The modelled peak

was 32.13mg/l. This produced a peak error of 114.2%, which corresponds to the maximum error obtained from the entire simulation.

Figure 14.5 shows the modelling of ammonia by GPS-X. The average modelled error was -10% and the maximum error was -27.5%. This error occurred at hour 25. The peak sampled concentration was 15mg/l at hour 95. The modelled peak was 14mg/l at hour 105. This produced an error with respect to the peak of - 6.6%.

Fig. 14.3 TSS in Activated Sludge Effluent (GPS-X - Storm)

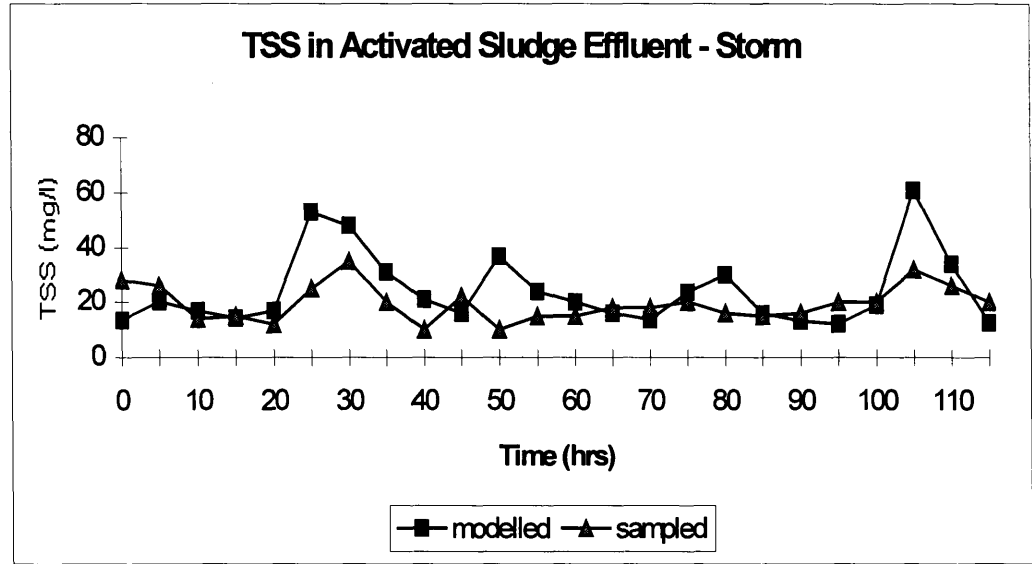


Fig. 14.4 BOD in Activated Sludge Effluent (GPS-X - Storm)

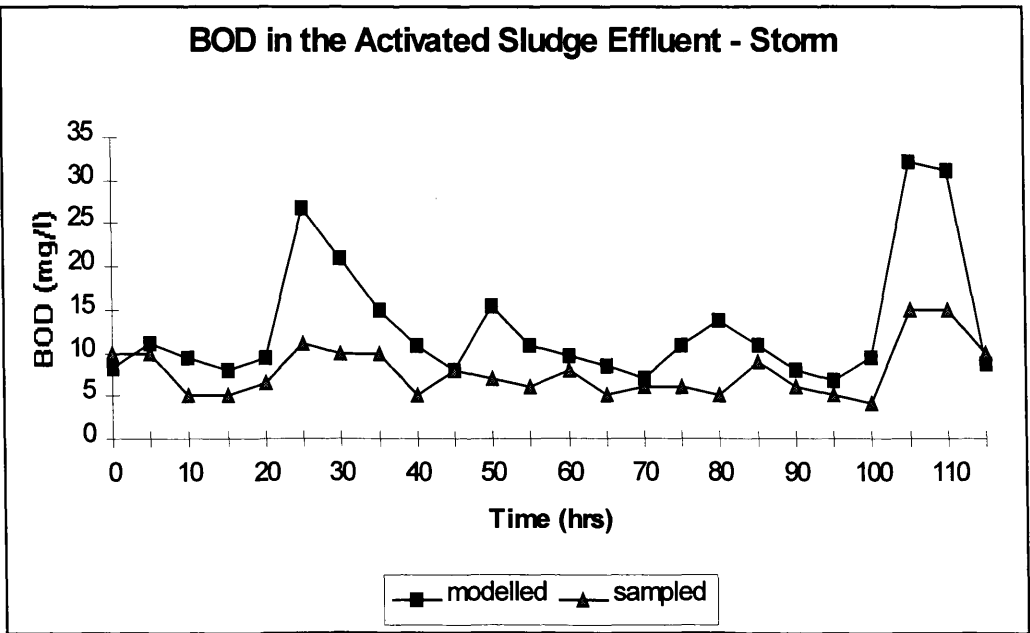
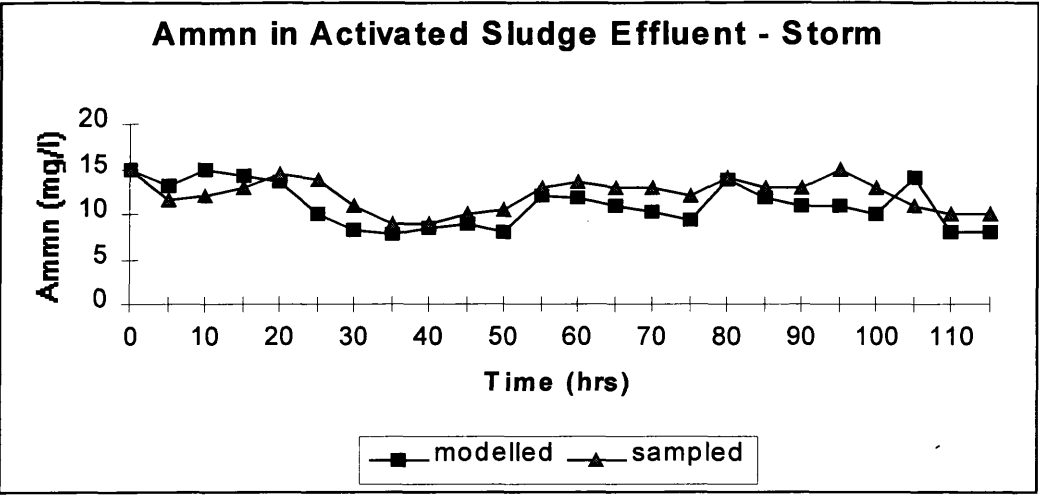


Fig. 14.5 Ammonia in Activated Sludge Effluent (GPS-X - Storm)



14.7 Storm Modelling Summary

The GPS-X model of the Sleepless Inch WTP over-predicted TSS and BOD at certain time steps. However, this problem was attributed to the difficulty in accurately representing the MLSS within the reactor. A loose control was set up with a set point of 1350 mg/l which allowed the suspended solids within the reactor to fluctuate between 1180 mg/l to 1650mg/l on the daily basis. This was similar to concentrations which were observed at Sleepless Inch. However, due to the lack of a ‘scientific’ MLSS control strategy at Sleepless Inch a problem lay in representing the MLSS concentrations at the beginning of the storm period. As MLSS is a critical modelling parameter this problem was considered to be responsible for the general over-predictions. As WRc experienced the same difficulties whilst constructing the STOAT model the hypothesis was reinforced (Bryan, 1993). This problem could not be rectified and therefore the results were deemed to constitute as well a calibrated model as feasible.

14.8 Comparison of STOAT and GPS-X

Figures E.19, E.20, 14.6 and 14.7 show a comparison between the GPS-X and the prototype version of the STOAT software. Table 14.3 and 14.4 also provide a summary of the errors obtained from the two software packages under storm weather conditions.

Table 14.3 Modelled Errors in Primary Tank Effluent – STOAT & GPS-X

Primary Effluent	TSS	BOD
Average Error – STOAT	+6%	+7%
Average Error – GPS-X	-4.5%	+4%
Peak Error - STOAT	+11.25%	+9.6%
Peak Error – GPS-X	+8.75	15.5%

Table 14.4 Modelled Errors in Activated Sludge Effluent – STOAT & GPS-X

Activated Sludge Effluent	TSS	BOD
Average Error – STOAT	+71%	+204%
Average Error – GPS-X	+32%	+69%
Peak Error - STOAT	+57.1%	+966.6%
Peak Error – GPS-X	+51.4%	+114.2%

14.81 Primary Tank Summary

Figures E.19, E.20 and table 14.3 show that both models, GPS-X and STOAT offer similar levels of accuracy for TSS and BOD, however STOAT over-predicts both determinands to a greater extent. Ammonia was not considered for this analysis as the WTP does not nitrify

14.82 Activated Sludge Tank

Figure 14.6, 14.7 and table 14.4 shows that TSS have also been modelled to a higher level of accuracy by GPS-X. It can be seen that both models over-predict BOD however with GPS-X the over-predictions were significant less. This was because the simplifying hydrolysis assumptions utilised in STOAT were not utilised in GPS-X. Consequently, GPS-X provided much better results. It can therefore be concluded from the above analysis that GPS-X was an overall better model.

Fig. 14.6 TSS in Activated Sludge Effluent (STOAT & GPS-X - Storm)

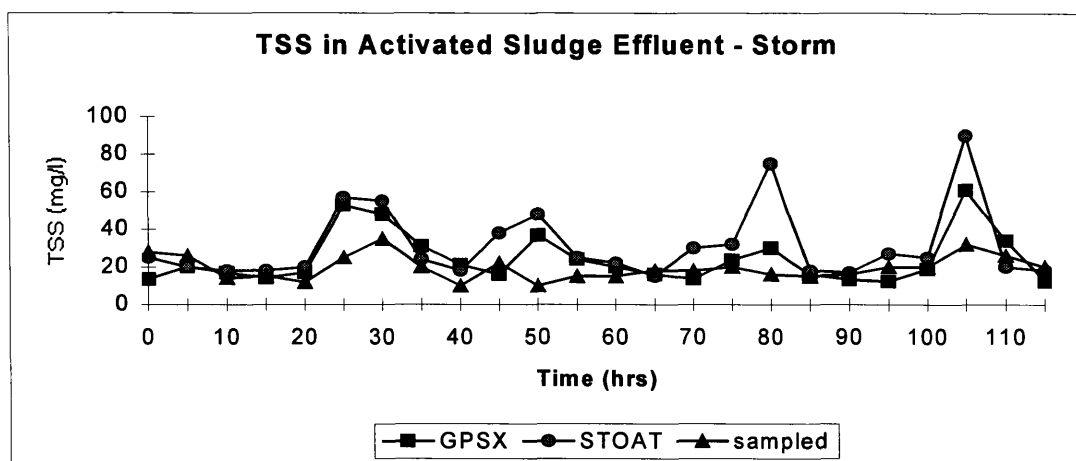
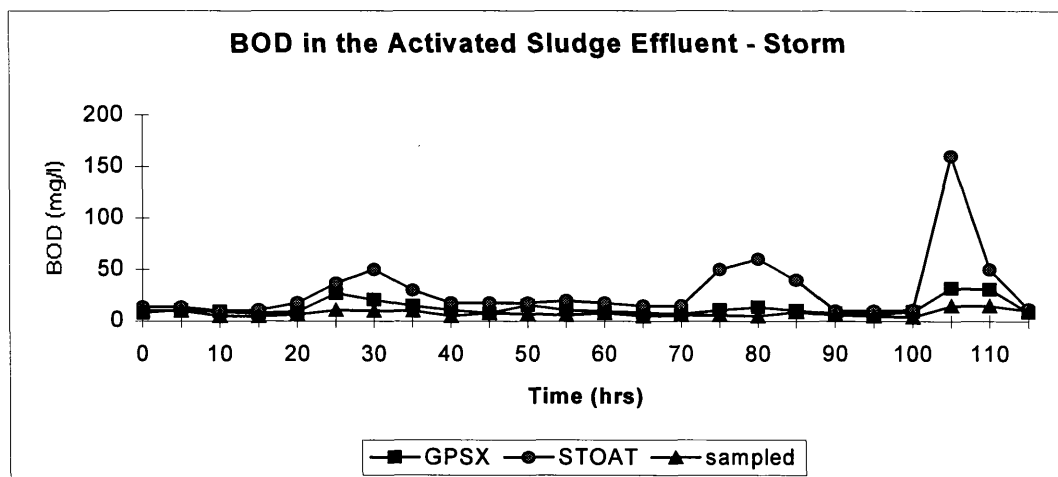


Fig. 14.7 BOD in Activated Sludge Effluent (STOAT & GPS-X - Storm)



14.9 Storm Tank Calibrations

As discussed in chapter thirteen the STOAT model was handed over by NoSWA to the University of Abertay Dundee in a semi-completed state with storm tank calibrations still requiring to be carried out. This work however was carried out using GPS-X as the above analysis showed this package to be the more appropriate of the two. In order to carry out this work an additional data collection exercise required to be undertaken as the previous data collection exercise did not provide suitable storm tank data. This subsequent data collection exercise provided data for two separate events - 26/10/95 and 31/10/95. The characteristics of these events are shown in table 14.5.

Table 14.5 Storm Tank Data Collection Appraisal

Date	Location and Number of samples	Rain Gauge Murray Royal (Peak and Vol.)	Rain Gauge Perth Grammar (Peak and Vol.)	Rain Gauge Burghmuir (Peak and Vol.)
26/10/95	Inlet	30 mm/hr	20 mm/hr	25mm/hr
	58	31.4 mm	30.6 mm	35mm
	Storm Tank Overflow			
	24			
31/10/95	Inlet	7 mm/hr	8 mm/hr	8mm/hr
	33	5 mm	3.8 mm	4.4mm
	Storm Tank Overflow			
	14			

As the storm tanks at Sleepless Inch are of the same design as the primary settlement tanks the calibration parameters required to calibrate the primary tanks were copied to the storm tanks. The modelled predictions are shown in figures E.21 to E.28. It can be seen from figure E.21 however, that the flow from the storm tank on the 26/10/95 has been poorly represented. The reason for the mismatch is attributable to operational problem which were encountered during the data collection exercise. It can be seen that the observed overflow begins approximately at 12.28, whereas the modelled flow is already spilling a significant amount at this time step. It can also be seen that the modelled flows tail away more quickly than observed flows. The reason for these anomalies were that the underflow valve from the storm tanks had been left open, and thus a significant portion of the influent to the tank was being returned directly to the plant inlet. This unfortunately went unnoticed for some time however, once the problem was noted the valves were closed, allowing the tanks to 'fill and spill'. As the model assumed the valves to be closed the modelled tank 'fills and spills' much sooner than the actual tank. Furthermore, the model was not returning these flows to the inlet of the plant and thus the flows were relieved from the system much quicker. This explains why modelled flows tail away quicker than observed flows. Due to these problems the total modelled volume was under-predicted by

46%. The qualitative data however, did not appear to be significantly affected (figs E.22, E.23 & E.24). The average modelled errors for TSS, BOD and Ammonia were calculated as +4.6%, +56% and +6%, respectively. No meaningful analysis could be carried out with respect to the peak concentrations due to the problems discussed above.

Figure E.25 shows improved modelling of flow from the storm tank overflow for the 31/10/95 event. The problems experienced during the 26/10/95 event did not occur here as the valve within the storm tank was closed throughout the duration of the storm event. The modelled error with respect to total volume was -16%. Figure E.26 shows the modelling of TSS in the storm tank effluent. The average modelled error was calculated as +14%. The observed peak concentration occurred at 11:04am and was 504mg/l. GPS-X predicted a concentration of 59mg/l to produce an error with respect to the peak of -88%. This happened to be the maximum error obtained from the entire simulation. Figure E.27 shows the modelling of BOD. Similar to TSS, BOD was also under-predicted at the beginning of the overflow operation, however the subsequent trend was one of more accurate modelling. The reason for this initial discrepancy is believed to be due to sewage standing within the tanks prior to the data collection exercise. This is very probable as rainfall records show that the WTP would have received storm flows 14hrs prior to this rainfall event. As the underflow valves from the tank were closed the tanks would not have emptied prior to the onset of the subsequent storm. This hypothesis is substantiated as the timing of the spill could only be represented accurately by defining the initial conditions of the storm tank as partially full. The average modelled error was calculated as +58%. The peak and maximum errors were +145% as both occurred at the same time-step (11:04am).

Figure E.28 shows the modelling of ammonia in the storm tank effluent. The average error was -4%. The maximum error was +10%.

14.10 WTP Modelling:- Overall Summary

From the analysis carried out it was concluded that the GPS-X model could represent the effluent quality under both dry and wet weather conditions to a

reasonable degree of accuracy for all unit processes under consideration. No problems were encountered during the calibration/verification process and no significant errors were evident in the modelled data. The prototype STOAT model was prone to over-predict BOD during periods of high hydraulic loading. Consequently, GPS-X was considered to be the most appropriate tool for this project.

Subsequent to this evaluation WRc released the full version of the STOAT software. This commercially available package gave greater consideration to the hydrolysis processes, which were responsible for the poor BOD predictions in the prototype model. The full version software was thus made available by WRc for use within this research project (evaluation purposes only) allowing a comparative analysis to be carried out between STOAT (prototype), STOAT (full version) and GPS-X. This work is detailed in Appendix C. Nevertheless, GPS-X was used as the WTP modelling platform for this research project.

Chapter Fifteen

WTP Sensitivity Testing

15.1 Introduction

As discussed in chapter 12 it was decided that hypothetical quality data should be utilised in preference to the data provided by the deterministic sewer flow quality model. Consequently, sensitivity testing of the WTP model was carried out using various influent flow and quality characteristics. The objective of these tests were to aid the preparation of the 'hypothetical' influent data and to provide information with regard to the errors which could be expected if 'actual' data differed from the developed hypothetical profile. This was considered to be important analysis as it allowed a level of confidence to be placed in the actual total emission analysis which was subsequently carried out (chapter twenty).

15.2 Sensitivity Test No.1

Determination of the Maximum Possible Input Data Time Step

The first test carried out was concerned with ascertaining the maximum WTP input data time step which could be used for simulations. The objective of this analysis was to minimise the long computational times associated with the complex modelling. The results from this analysis would show the maximum input data time step which could be utilised without compromising the accuracy of the WTP output results. The results would also minimise the preparatory time requirements associated with the generation of the hypothetical quality data.

15.21 Dry Weather Testing

In order to ascertain the sensitivity of the model to input time step interval under dry weather flow conditions four different scenarios were utilised:-

- i) Input data at 5 minute intervals

- ii) Input data at 10 minute intervals
- iii) Input data at 20 minute intervals
- iv) Input data at hourly intervals

Figures 15.1 and 15.2 show the flow and pollutant characteristics used in the tests. The characteristics were obtained from the Perth data collection exercise (McGregor, 1995).

Fig. 15.1 WTP DWF Influent Profile

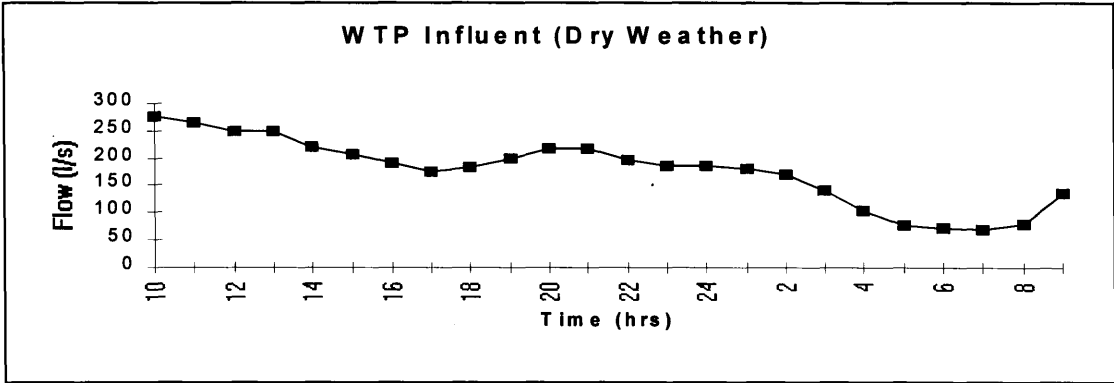


Fig. 15.2 WTP DWQ Influent Profile

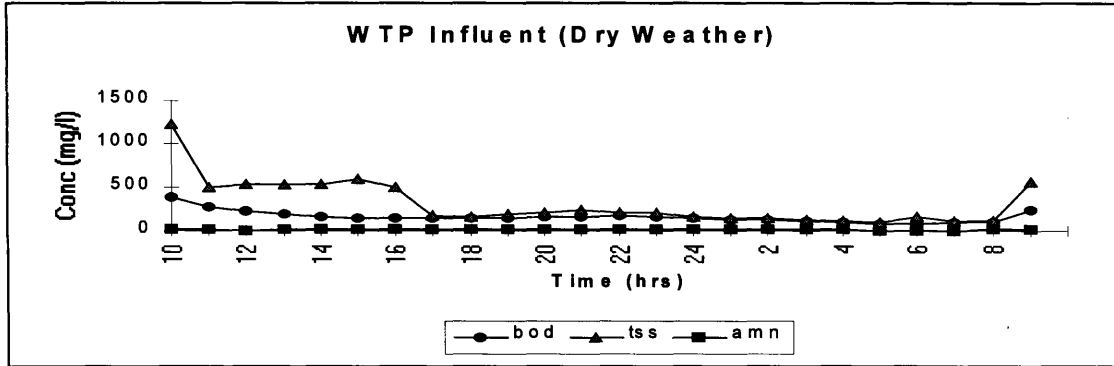
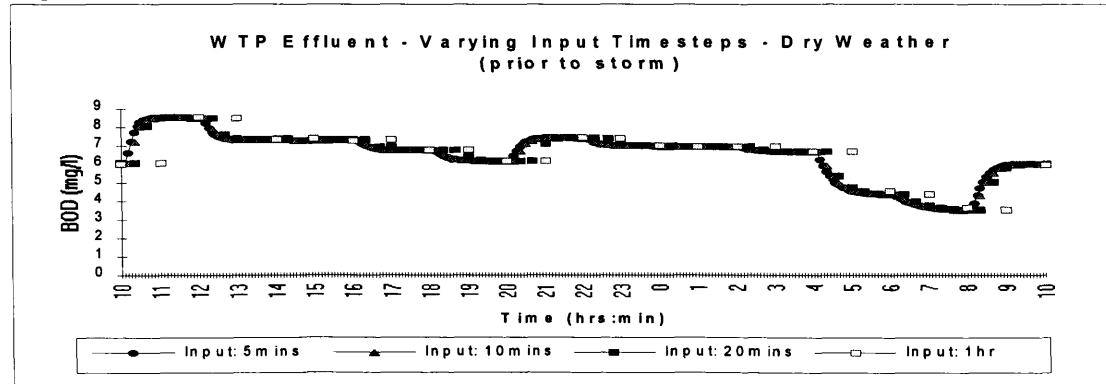


Figure 15.3 shows the modelled WTP effluent data for the four different scenarios.

Fig. 15.3 WTP BOD Effluent Profiles for Test Scenarios



It can be seen from figure 15.3 that very little difference exists with the timing of the BOD peaks and troughs with respect to the input intervals of 20 minutes and 1 hour, and that, in general the effluent concentrations were very similar for all four tests. This was expected as the influent characteristics of the wastewater do not vary greatly over hourly intervals under dry weather conditions.

15.22 Wet Weather Testing

Combined influent, generated from a hypothetical rainfall event (Fig. 15.4), was used to test the sensitivity of model under wet weather conditions.

Fig. 15.4 Hypothetical Rainfall Profile

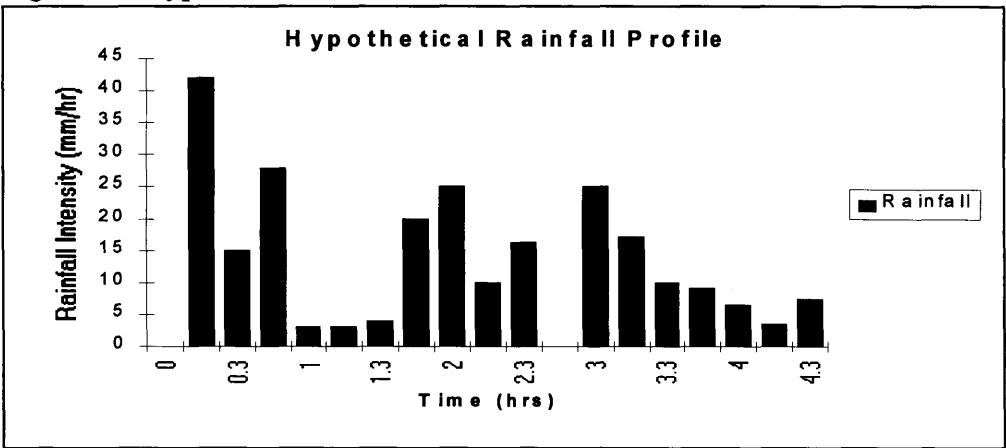


Figure 15.5 shows the resulting hydrograph which was obtained from the Hydroworks hydraulic model:-

Fig. 15.5 Influent Flow Data for WTP Model (Storm)

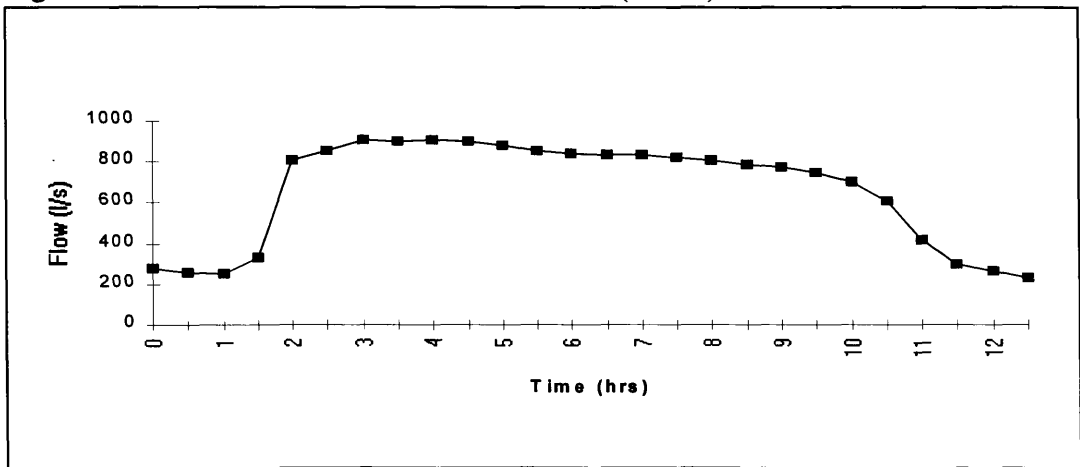
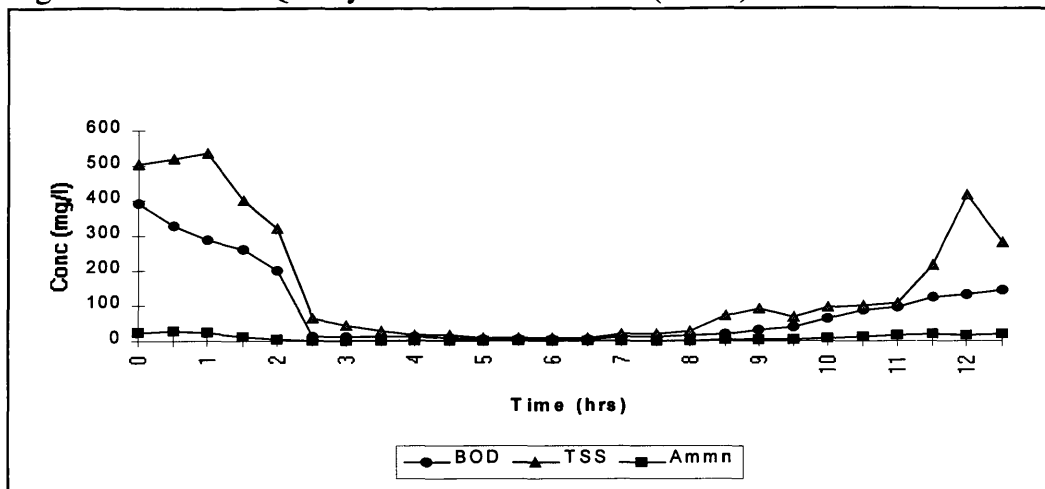


Fig. 15.6 Influent Quality Data for WTP Model (Storm)



The quality data as shown in fig 15.6 were obtained from the Hydroworks -QM model. Although it had been previously established that -QM would not produce accurate quality data (under combined loading conditions) this was not of importance with respect to this test. The reason being that the test objective was simply to ascertain whether the *same* influent profile would produce different output results if varying input time steps were used. Consequently, the identification of an accurate influent profile was not of critical importance.

15.23 Test Procedure

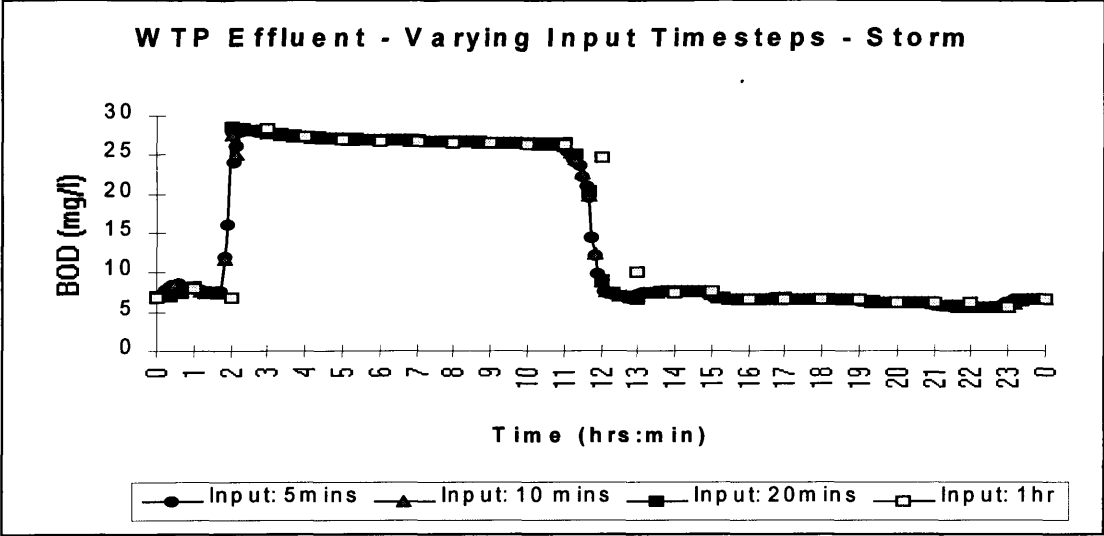
The following four scenarios were initially tested:-

- i) input data at 5 minute intervals
- ii) input data at 10 minute intervals
- iii) input data at 20 minute intervals
- iv) input data at 1hr intervals

15.24 Wet Weather Test Results

Figure 15.7 shows the effluent profiles from the WTP for the respective scenarios.

Fig. 15.7 WTP Effluent Profiles from Input Interval Analysis Tests



It can be seen from figure 15.7 that the effluent profiles have not been affected by the respective input time steps. A slight difference exists with regard to the timing of the peaks and troughs for the one hour input interval scenario however, the overall trend of the profile is the same. Table 15.1 compares the loads discharged from the WTP for the respective simulation scenarios.

Table 15.1 WTP Effluent Load - BOD

	WTP Effluent Load – BOD			
	Input Interval:- 5 minutes	Input Interval:- 10 minutes	Input Interval:- 20 minutes	Input Interval:- 1hour
Effluent load (storm 1-12hrs)	440Kg	439Kg	439Kg	430Kg
Effluent load (dwf 12-14hrs)	76Kg	76Kg	76Kg	74Kg
Total load (0 -24hrs)	516Kg	515Kg	515Kg	504Kg

It is apparent that the load discharged from the WTP is only marginally decreased by the one hour time step interval. Due to this insensitivity, a further scenario was carried out to ascertain the model’s response to a two hour input interval. These results are presented in table 15.2:-

Table 15.2 WTP Effluent Loads (Varying Input Time step Intervals)

	Input Interval:- 5 minutes	Input Interval:- 1 hour	Input Interval:- 2 hours
Effluent load (storm 1-12hrs)	440Kg	430Kg	408kg
Effluent load (dwf 12-14hrs)	76Kg	74Kg	70.5kg
Total load (0-24hrs)	516Kg	504Kg	478Kg

It can be seen that when the two hour input interval has been utilised the effluent load is noticeably less than the loads resulting from the scenarios with smaller input time steps. Table 15.3 compares the percentage difference in effluent loads for the above analysis:-

Table 15.3 WTP Effluent Load Analysis (BOD)

Input Interval	WTP Effluent load (Kg/BOD/day)	% difference
Five minutes	516	
Ten minutes	514	-0.212
Twenty minutes	514	-0.212
One hour	504	-2.34
Two hours	478	-7.23

Table 15.3 shows that the 2 hour input interval scenario produced only a -7.23% load difference relative to the five minute input interval scenario. As the expected accuracy of the logged flow data is +/- 20% (with larger errors being possible from sewer flow quality data collection), it was concluded that a difference of -7.23% was acceptably small. Similar trends were observed for TSS and Ammonia as shown in tables 15.4 and 15.5.

Table 15.4 WTP Effluent Load Analysis (TSS)

Input Interval	WTP Effluent Load (Kg/TSS/day)	% difference (relative to 5 minute input interval)
five minutes	897	
1 hour	872	-2.7%
2 hours	836	-6.8%

Table 15.5 WTP Effluent Load Analysis (Ammonia)

Input Interval	WTP Effluent Load (Kg/Ammn/day)	% difference (relative to 5 minute input interval)
five minutes	302	
1 hour	295	-2.3%
2 hours	289	-4.3%

Consequently, it was hypothesised that a two hour input interval would be suitable for WTP modelling. Technical support from Cambridge Control (software company) confirmed the validity of a two hour input data interval, as no justifiable benefit would be achieved using a smaller time step for WTP analysis (Sedarity, 1996). This opinion was shared by Guderian et al, (1997). Regardless of this advice however, a further test was carried out to ascertain whether greater errors would result if a significant flush were to occur. This test was carried out because it was believed that an input interval of two hours would be too gross to adequately define the pollutant characteristics of the flush. As it was necessary to develop hypothetical flush profiles which, to prevent underestimation of WTP emissions, would require to be conservatively large, it was considered important to ascertain the error which would result if these conservative profiles were defined using a two hourly input interval. From observations of the inlet WTP data (McGregor, 1995) it was hypothesised that the maximum "flush" concentrations, for an influent flow of between 900-1000l/s (maximum WTP inflow) could be taken as 800:650 mg/l (TSS:BOD). Utilising input time steps of five minutes, 1 and 2 hours, the sensitivity of the model to this flush profile was analysed. The results are shown in table 15.6 below:-

Table 15.6 Five Minute, One Hour and Two Hour Input Interval Analysis (TSS)

Input Interval	WTP Effluent Load (Kg/TSS/day)	% difference
5 minutes	946	
1 hour	896	-5.28%
2 hours	1070	+13%

It can be seen from table 15.6 that the two hour input interval caused an effluent loading difference of 13% (relative to the five minute interval) for the scenario where the hypothetical flush was applied. Table 15.7 shows a 25% error for BOD.

Table 15.7 Five Minute, One Hour and Two Hour Input Interval Analysis (BOD)

Input Interval	WTP Effluent Load (Kg/BOD/day)	% difference
5 minutes	531	
1 hour	517	-2.63%
2 hours	663	+25%

This additional analysis has therefore shown that if a large flush were to occur, the TSS and BOD differences which could result as a consequence of utilising a two hour input interval could increase to 13 and 25%, for TSS and BOD respectively. However, utilising the one hour input interval it can be seen that the resulting errors were significantly lower (<7%). Again this trend would be expected as the one hour input interval, although still crude, allows the flush characteristics to be defined more accurately. For these reasons it was decided to utilise the one hour input interval for the modelling.

Furthermore, due to the extensive computational times associated with sewer hydraulic modelling it became apparent that the WRc SIMPOL model (hydraulic) would require to be utilised. This model is described in more detail in chapter nineteen. However, the important aspect concerning SIMPOL is that the output data are provided on an hourly basis. Consequently, for modelling compatibility it was decided that an hourly input interval should also be used for the WTP modelling.

15.3 Sensitivity Test No.2

WTP Sensitivity to Varying Flush Profiles

It was noted from sensitivity test No. 1 that where hourly input data time step intervals were utilised no significant difference existed between WTP effluent

loads for the scenario where no flush occurred and for the scenario where the hypothetical flush was applied. This is demonstrated more clearly in tables 15.8 and 15.9.

Table 15. 8 TSS Effluent Load Comparison (1 Hour Input Time-step Interval)

	Original profile TSS (Kg/TSS)	Adjusted profile TSS (Kg/TSS)	Difference
0-12hrs	748	771	+ 3%
12-24hrs	124	125	+ 1 %
Total Load (24hrs)	872	896	+ 2.75%

Table 15.9 BOD Effluent Load Comparison (1 Hour Input Time-step Interval)

	Original profile BOD(Kg/TSS)	Adjusted profile BOD (Kg/BOD)	Difference
0-12hrs	430	443	+3%
12-24hrs	74	74	0
Total Load (24hrs)	504	517	+2.5%

It can be seen that where the adjusted flush profile was utilised only a 3% difference in load resulted (TSS and BOD). It was initially believed that the inability of the sewer flow quality models to predict the flush would constitute a major problem with respect to the integrity of the WTP results. However, the above analysis surprisingly showed this not to be the case. As these tests had been carried out using the maximum WTP influent flow rate, it was apparent that the WTP would be even less sensitive to the flush characteristics at lower flow rates. Consequently, further tests were carried out to ascertain whether a standard flush profile could be used with any given influent hydrograph.

Additional assumptions were therefore made with regard to maximum flush profiles which could result from flows of lesser magnitude than 1000l/s. These assumptions are summarised below and are based on the original assumption that a flow rate of between 900 and 1000l/s could give rise to a maximum “flush” concentration of 800:650 mg/l TSS:BOD.

15.31 Additional assumptions

Flows of up to 400l/s could give rise to a maximum flush of 550:450mg/l TSS:BOD.

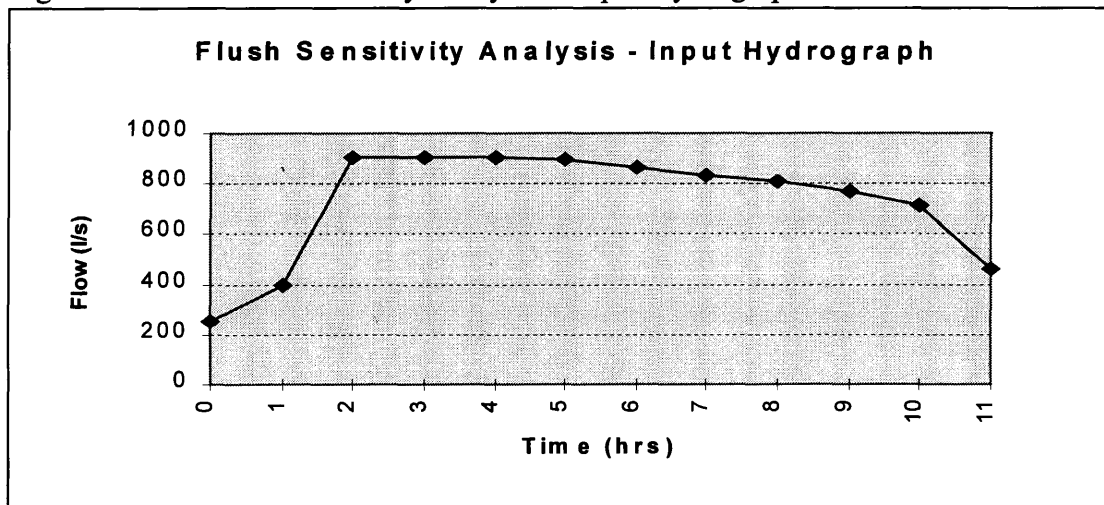
Flow rates between 400 and 600l/s could give rise to a maximum flush of 650:500 mg/l TSS:BOD.

Flow rates in excess of 600l/s could result in the maximum possible flush concentration of 800:650mg/l TSS: BOD

15.32 Test Procedure

Comparisons were made between the effluent profiles which resulted using the above flush characteristics and the profiles which resulted when the maximum flush concentration of 800:650 mg/l TSS:BOD was applied for all ranges of flow. The input hydrograph utilised in the tests is shown in figure 15.8.

Fig. 15.8 Flush Sensitivity Analysis - Input Hydrograph



The flow rate at time step one hour, corresponds to the time step where the flush characteristics were applied. This is the only time step in the combined event where the data (flow and quality) were adjusted, thus allowing the sensitivity of (only) the flush to be analysed. The input data (hydraulic and qualitative) are shown in table 15.10 with the exception of the time step at hour one, where the variables x, y and z (table 15.10) were varied for the different test scenarios.

Table 15.10 WTP Input Data

Time (hours)	Flow (l/s)	Concentration:- TSS : BOD (mg/l)
0	256	240 : 201
1	x	y:z
2	907	100 : 43
3	906	65 : 14
4	905	44 : 11
5	900	18 : 11
6	863	10 : 6
7	835	20 : 12
8	812	32: 14
9	773	92 : 33
10	715	101 : 68
11	458	110 : 98

As it was assumed that a flow rate of between 400 and 600l/s could give rise to a flush of 650:500mg/l TSS:BOD, the first comparison was carried out to ascertain the difference which would result if the actual flush which occurred was 800:655 (assumed max. possible flush concentration). As the WTP would be most sensitive to the higher flow rate within this range (due to the reduced residence time) the test was carried out with a flow rate of 600l/s.

15.33 Test Results

Fig 15.9 Flush Sensitivity Analysis at 600l/s - WTP Effluent TSS

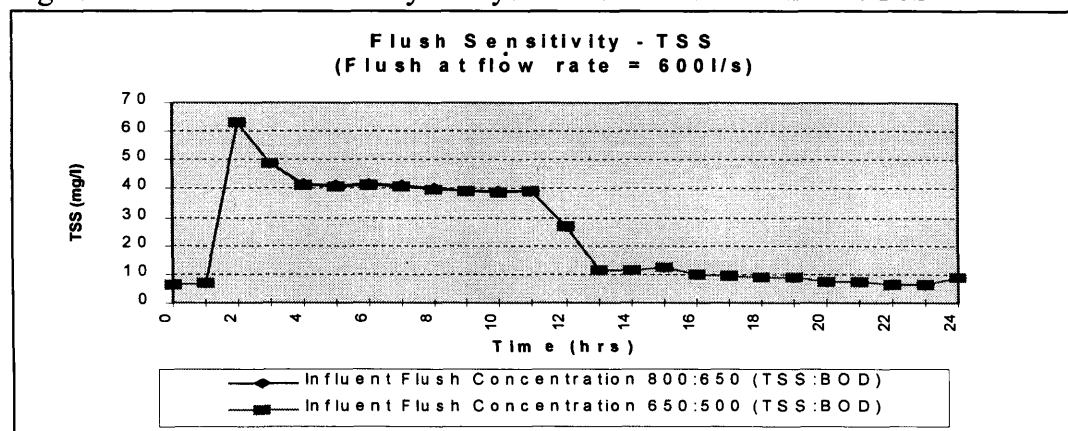
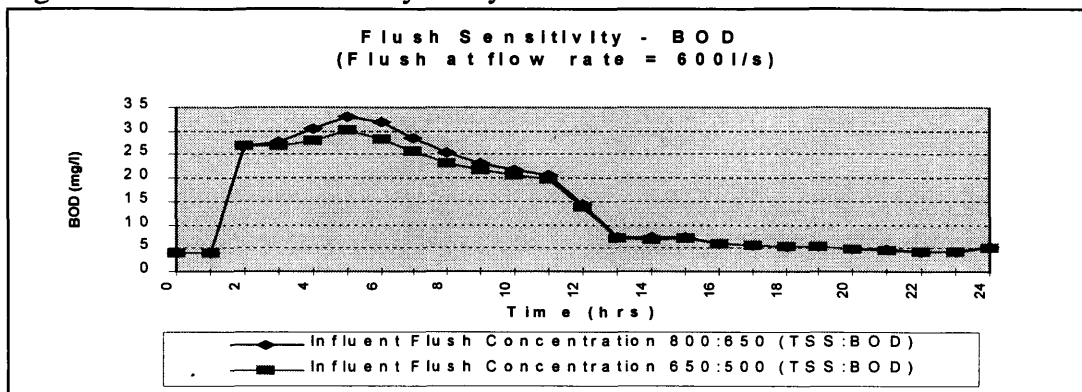


Fig 15.10 Flush Sensitivity Analysis at 600l/s - WTP Effluent BOD



Figures 15.9 and 15.10 show very little difference between the effluent profiles utilising an input flush of either 650:500 or 800:650 (TSS:BOD) at 600l/s. Due to the lack of sensitivity it was not necessary to repeat the test at the lower flow rate within the range as the sensitivity would be even less. Consequently, it was concluded that the utilisation of the maximum flush of 800:650 could be applied to flows within the 400 - 600l/s range as the WTP models outputs are largely insensitive to the change.

As it had also been assumed that combined flows of up to 400l/s could produce a maximum flush of 550:450 the analysis was repeated to ascertain the difference which would result if the flush were increased to 800:650. Again this test was carried out at the maximum flow rate within the defined range (400l/s) to ensure the maximum sensitivity was being observed. The results are shown in figures 15.11 and 15.12.

Fig. 15.11 Flush Sensitivity Analysis at 400l/s - WTP Effluent TSS

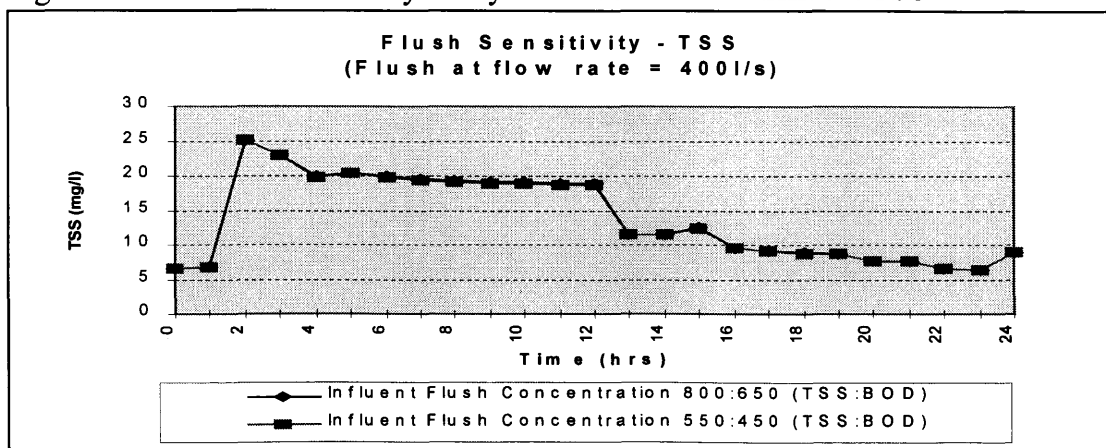
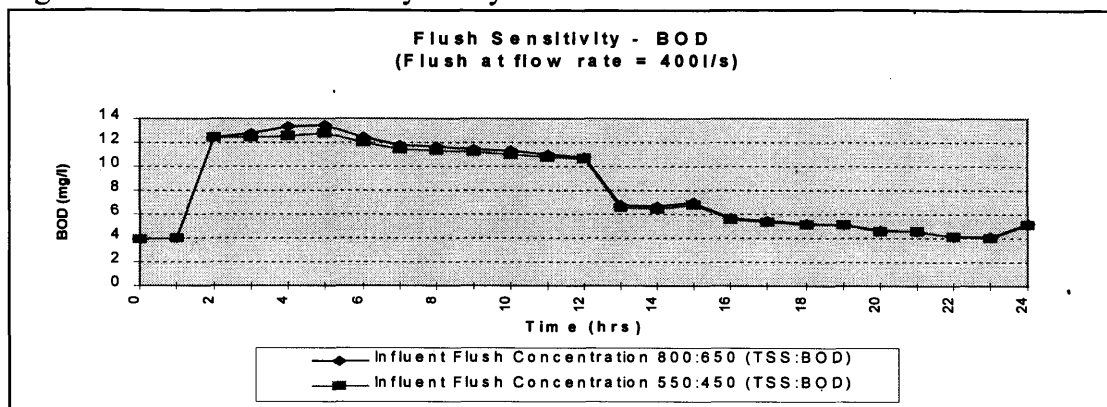


Fig. 15.12 Flush Sensitivity Analysis at 400l/s - WTP Effluent BOD



It can be seen from figures 15.11 and 15.12 that the model is largely insensitive to the respective flush concentrations.

15.4 Conclusion for Sensitivity Test No.2

The analysis has provided positive evidence that a standard flush of 800:650 mg/l (TSS:BOD) can be used for all ranges of flows as the output results are not adversely affected by changing these concentrations. This greatly eased the practical problem of the sewer flow quality modelling deficiency.

15.5 Sensitivity Test No.3

Definition of Diluted Pollutant Concentrations for Remainder of Combined Flows

Although the sewer flow quality model was deficient in terms of its ability to predict the magnitude of the flush, it was initially believed that it could still provide meaningful information with respect to the diluted wastewater characteristics which occur after the initial stages of combined flows. This was considered to be a major strength of the sewer flow quality model as these diluted components can last for long periods (especially if in-sewer storage is utilised) and cause significant disruption at the plant. However, from observations of -QM outputs and observations of collected data (McGregor, 1995) it was apparent that -QM could also be expected to significantly over-dilute the diluted wastewater characteristics. The reason for this was that -QM dictated that all determinands could be diluted to less than 10mg/l, whereas data collection

suggested that TSS concentrations would not be reduced below 100mg/l and BOD concentrations would not be reduced below 20mg/l. Consequently, to prevent an illusion of accuracy it was decided to develop hypothetical pollutant profiles for the diluted component of the waste waters as well as the flush. Sensitivity test were therefore carried out to determine the sensitivity of the WTP model to varying diluted component concentrations. The objectives of these tests were to develop information with regard to the magnitude of the error which would result if the pollutants in reality deviated from those proposed via the hypothetical profile. Although these errors would require to be accepted whether large or small (as no suitable alternative method was available) the tests were carried out to ascertain the degree of confidence which could be given to the results. The main difficulty with respect to the development of these profiles was the definition of an acceptable range of influent concentrations. This was a consequence of the limited data which had been collected at the inlet of the WTP. The most useful data which were available for this analysis proved to be from the event collected on the 26/10/95 as this event produced substantial combined flows (900l/s +) for a period in excess of 24 hours. Associated quality data were obtained over a period of 4 hours thus providing significant information with respect to the degree of dilution which these high flows would provide. Another substantial rainfall event occurring on the 31/10/95 also produced a substantial duration of combined loading to the WTP (4 hrs). However, the most comprehensive data were obtained from the aforementioned event. Other data which were used in the development of the profiles were the data collected by NoSWA for the calibration of the prototype STOAT model. These data were collected over a twelve day period, however, none of the rainfall events in this data series produced a volume of runoff comparable to the event of the 26/10/95. In order to carry out this work, upper bound and lower bound assumptions required to be applied to the qualitative data. The upper bound assumptions/limits were obtained from the collected data set 26/10 as this event showed the degree of dilution which full hydraulic loading would provide. The lower bound assumption were that when the flow rate rose above DWF, the associated pollutants, after the flush, could not be greater than the average DWF

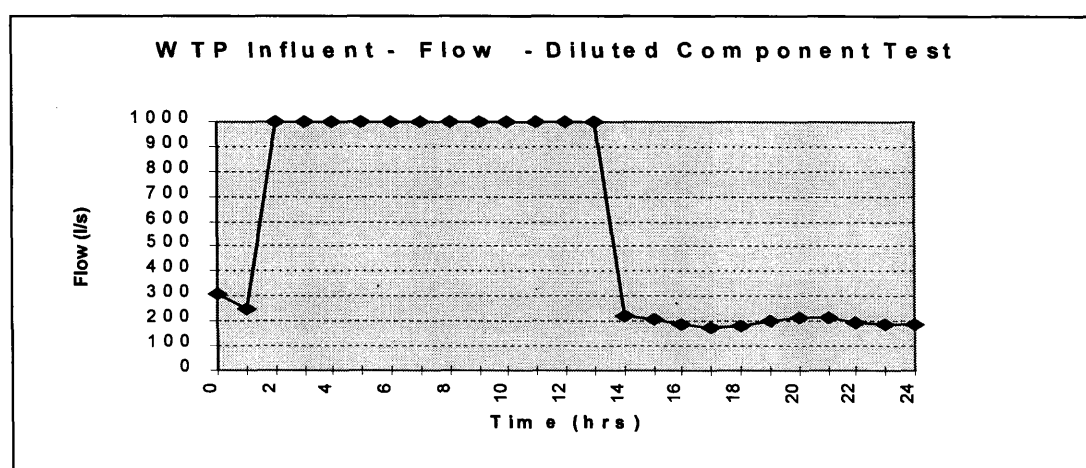
concentrations i.e. a dilution of dry weather flow would occur. These average concentrations were defined from data collection (McGregor. I, 1995) as 205:170 mg/l (TSS:BOD).

Three different pollutant characteristics were used for this test:-

- a) Those indicated by the sewer flow quality model (30:10 TSS:BOD mg/l)
- b) Those indicated by data collection (120:25 TSS:BOD mg/l)
- c) Average DWF pollutant concentrations (205:170 TSS:BOD mg/l)

The hydrograph which these concentrations were applied to is shown in figure 15.13 .

Fig. 15.13 WTP Influent Flow Profile – ‘Dilution Analysis’



It can be seen that the hydrograph was based on the hydrograph utilised in the flush sensitivity analysis, however a continuous flow rate of 1000l/s was sustained for the entire duration of the event. By comparing the results from test a) above with those obtained from test b) above an evaluation could be made with respect to the error which would have resulted if the sewer flow quality model had been used in the total emission analysis. By comparing the results from test c) with those from test b) the evaluation of the potential error which could result if the actual concentrations varied from those defined in the derived hypothetical profile could be analysed. As the data defined in test c) correspond

to average dry weather pollutant concentrations the evaluation would show the maximum error which would result. This is because this scenario represents the hypothetical case where the storm flows do not dilute the dry weather flows. As a dilution would always result the comparison would show the maximum difference which could possibly result and thus the upper bound of the potential error.

The initial tests were carried out for TSS and BOD. As ammonia is not related to either determinand its analysis has been documented later for clarity.

The results are shown in figure 15.14:-

Fig.15.14 Dilution Analysis at 1000l/s - WTP Effluent (TSS)

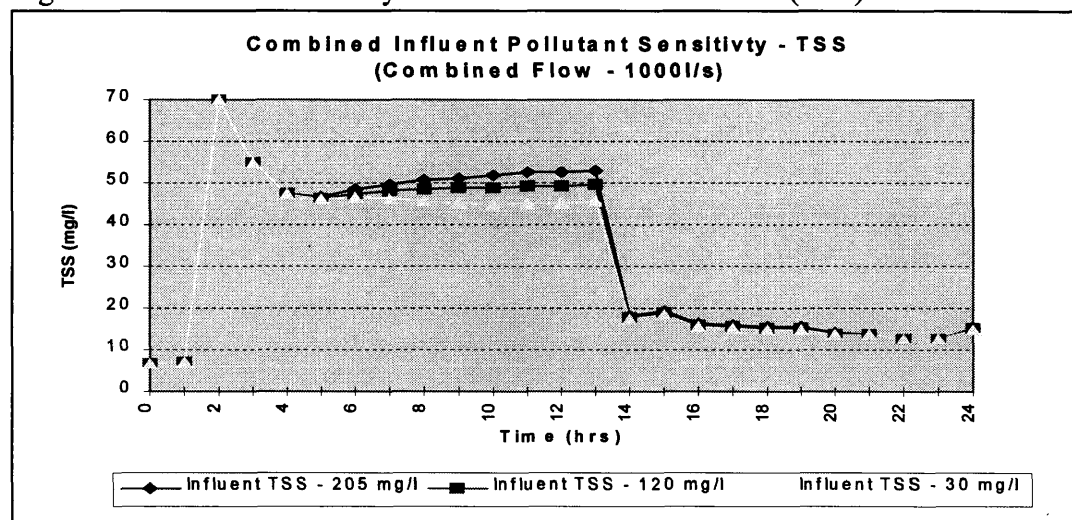


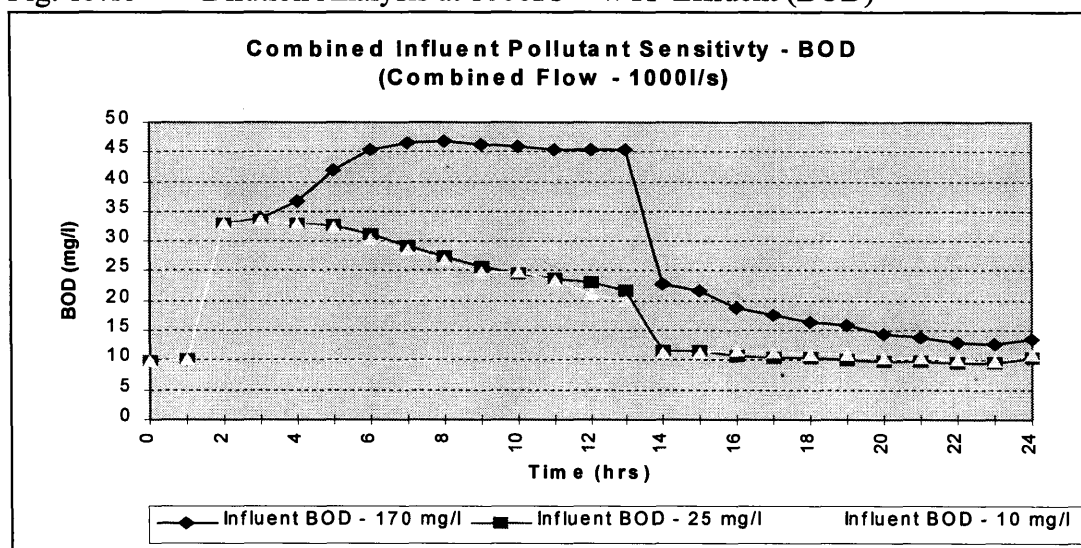
Figure 15.14 shows the insensitivity of the TSS outputs to the various influent concentrations. The maximum difference between the three tests occurred at hour 13 and was 17%. Consequently, if the influent data were to vary either above or below 120mg/l (for other events which also produce flow rates of 1000l/s) it would not result in a significant error. Furthermore, as this is the maximum flow rate which the plant would be subjected to, the analysis has shown the maximum sensitivity. This is demonstrated by the following tests. It is interesting to note that if the sewer flow quality model were used to provide input data to the WTP it would not cause the WTP model to produce significant modelling errors for TSS.

15.51 Conclusion

The TSS influent data can therefore confidently be taken as 120 mg/l (as suggested by the data collection exercise) for flow rates of approximately 1000l/s.

The BOD analysis is shown below:-

Fig. 15.15 Dilution Analysis at 1000l/s - WTP Effluent (BOD)



It can be seen that BOD has proved to be a more sensitive parameter than TSS as the output results have shown sensitivity to the varying influent characteristics. The difference between the influent characteristics of 10mg/l and 170mg/l was 125% at hour 13. This greater sensitivity was however expected as more complex WTP processes are required for the removal of BOD.

Consequently, the tests have shown that greater care requires to be taken over the definition of the influent BOD profiles. However, this problem is greatly simplified by the trend which BOD takes under combined loading conditions. As previously discussed in section 15.4 the data collection exercise showed that BOD can be expected to be heavily diluted by the combined flows. This restricts the possibility of widely varying influent concentrations being applicable, which in turn diminishes the possibility of significant modelling errors occurring. The reason for this is that it cannot be expected that the hypothetical profile would be significantly different from the actual profile if both exhibit heavy dilution. This is demonstrated by the above figure which shows that only a marginal difference

existed between the BOD effluent profiles for the scenarios utilising dilution, i.e. the influent concentrations corresponding to those defined by the data collection exercise and those defined by the sewer flow quality model. The error in this case at hour 13 was only 23%.

Consequently, it was concluded that a BOD influent profile of 25 mg/l (as suggested by the data collection exercise) can be utilised for all events where the hydraulic inputs are approximately 1000l/s as the error which would result if the actual concentration deviated from this value would not be significant (because the deviation itself would not be expected to be large).

It was also interesting to note that, similar to the TSS analysis, if the sewer flow quality model were used to provide BOD input data a large WTP modelling error would not be expected.

15.52 Summary of TSS/BOD Analysis for Test No.3 (at 1000l/s)

The analysis has shown that the effluent TSS concentrations would not be affected if the hypothetical profile deviated from the concentrations which will occur in reality. This is a consequence of the good settlement efficiency of the clarifiers at the Sleepless Inch WTP.

BOD in the effluent proved to be more sensitive to influent concentration however, it was established that if educated attempts were made at defining the influent concentration the potential error which would result if the hypothetical profile varied from reality would also not be expected to be large.

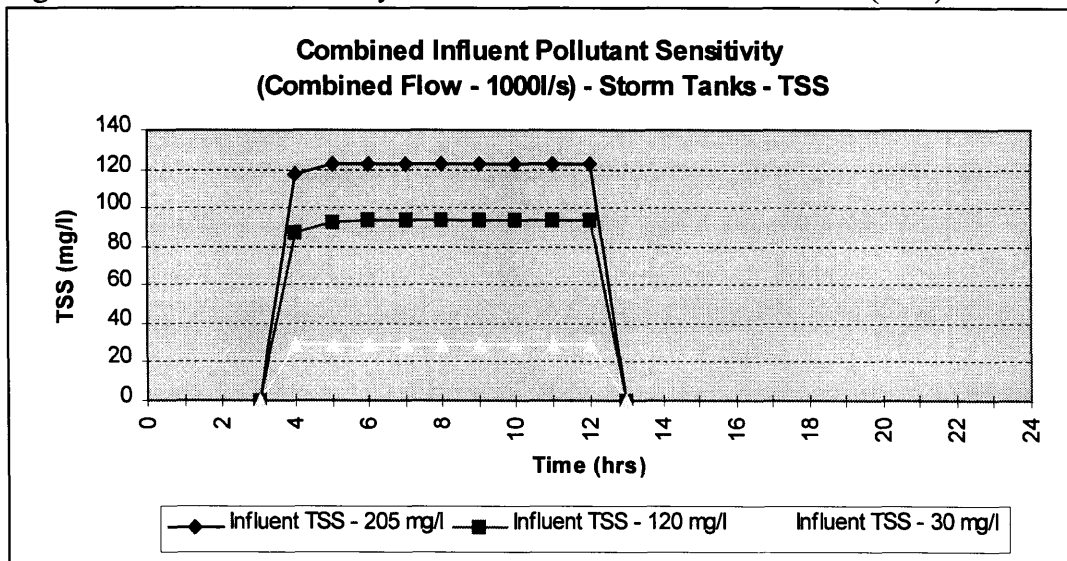
It was therefore apparent that if the sewer flow quality model were used to provide the WTP input data no significant error would result for either determinand, TSS or BOD. It was however, preferred to utilise hypothetical profiles in an attempt to keep the influent concentrations as close to reality as possible. The reason this was deemed necessary was that as the data collection exercise showed that TSS influent concentrations were likely to remain around 100 mg/l, it was possible that prolonged loading at such a concentration could result in dangerously high sludge blanket levels over a period of time. This would go unnoticed if the sewer flow quality model, which dictated significantly lower concentrations, was utilised.

As sewer flow quality model prediction for BOD (<10mg/l) were also lower than those suggested by the data collection exercise (not less than 20mg/l) it was possible that excessive biomass decay could result as a consequence of the unrepresentative influent concentrations, thus resulting in poorer WTP performance over time. Consequently, it was deemed more appropriate to utilise hypothetical sewer quality data as an attempt could be made to ensure the input data were as accurate as possible.

15.53 Storm Tanks

As the above analysis had only been concerned with the effluent from the WTP the test was repeated to ascertain the sensitivity of the storm tanks with respect to the varying influent concentrations. The results are shown in figure 15.16:-

Fig. 15.16 Dilution Analysis at 1000l/s - Storm Tank Effluent (TSS)

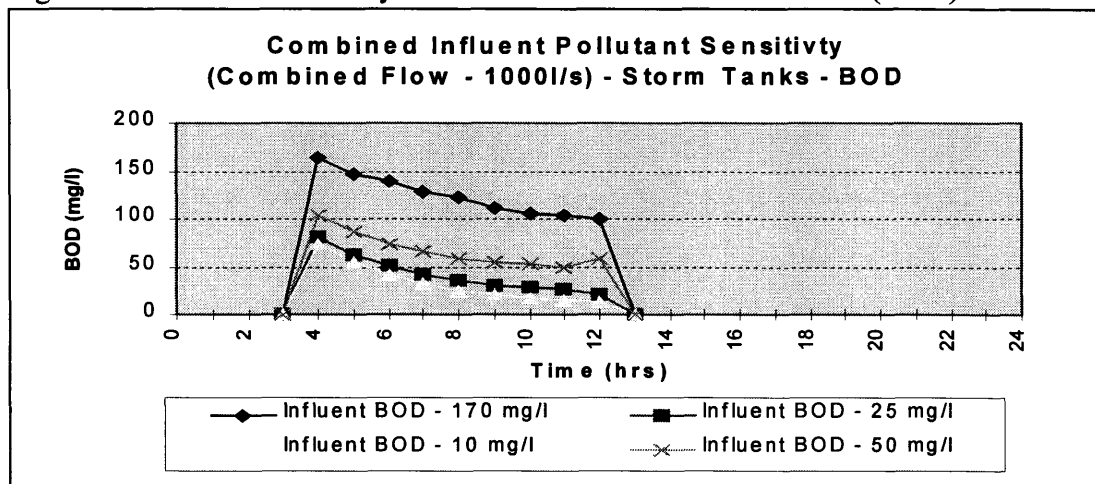


It can be seen that the storm tanks proved more sensitive to influent TSS concentrations than did the WTP. The average difference between the extreme influent characteristics (205mg/l and 30 mg/l) was 306% between hours 4 and 12. The reason for this greater sensitivity was that the TSS concentrations in the effluent from the WTP were a consequence of the sewage passing through two settlement processes. The storm tank is the first unit process which the solids pass through and consequently a greater sensitivity is apparent.

Figure 15.16 therefore shows clearly that if the sewer flow quality model was used to provide input data a reasonably large error would result in the storm tank

effluent. If a hypothetical profile was utilised the error could be minimised as influent TSS concentrations remain high under combined loading (around 120 mg/l).

Fig. 15.17 Dilution Analysis at 1000l/s - Storm Tank Effluent (BOD)



The results from these tests show the expected trend for BOD effluent quality. The trend generally being that the effluent BOD concentration from the tank is of a similar concentration to that which goes into the tank, albeit except for the initial effluent concentrations which are affected by a mixing with the highly polluted flush component. Even although, the influent BOD trend is one of heavy dilution the analysis has shown that the effluent concentration from this tank can be significantly erroneous if the chosen influent pollutant concentration is different from the concentrations which occur in reality. i.e. if the actual influent BOD concentration which occurred for an event was 50 mg/l and a concentration of 25 mg/l had been chosen for modelling purposes an error of approximately 50% would be expected to result from the effluent of the storm tank. This therefore shows that the storm tanks show greater sensitivity than the effluent from the WTP. Although the expected BOD dilution restricts the possibility of excessively erroneous influent concentrations being defined, the problem of accuracy still exist. This cannot be avoided and is a result of the poor sewer flow quality modelling tools which are available. Consequently, the definition of an accurate as possible influent profiles is important to ensure accurate WTP modelling results. This therefore suggests that hypothetical influent profiles would be more appropriate rather than utilising the results obtained from the

substandard sewer flow quality model. It was therefore concluded that influent concentrations of 120:25 mg/l (TSS:BOD) would be suitable for events generating flow rates of 1000l/s.

15.54 Sensitivity of WTP and Storm Tanks with Decreasing Flow Rate

As the above tests were carried out at the maximum flow rate the sensitivity of the effluent to the varying influent characteristics was at its greatest. Subsequent test were carried out to demonstrate the decrease in sensitivity which would result as a consequence of decreasing flow rate. The following section details only the results from the WTP effluent as the same general trend was observed for the storm tanks. The results presented below correspond to the tests which were carried out for flow rates 600l/s, 500l/s and 400l/s. The performance of the WTP at 600 l/s is in figures 15.18 and 15.19.

Fig. 15.18 Dilution Analysis at 600l/s - WTP Effluent (TSS)

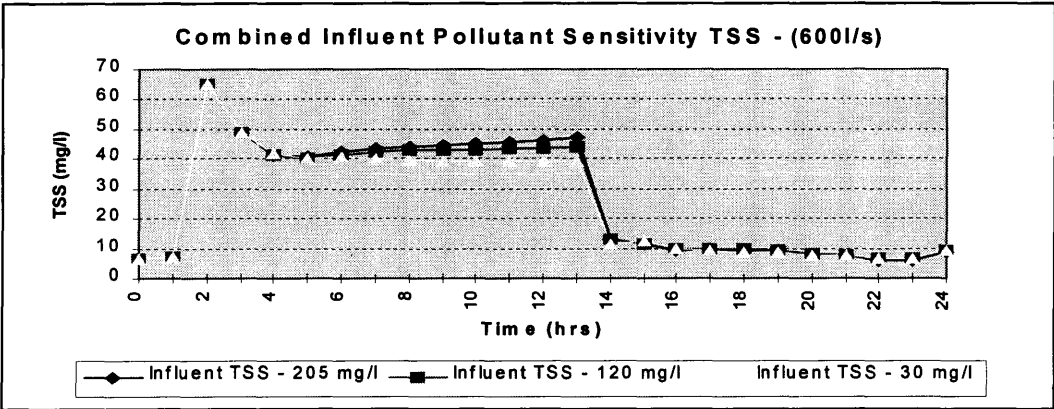
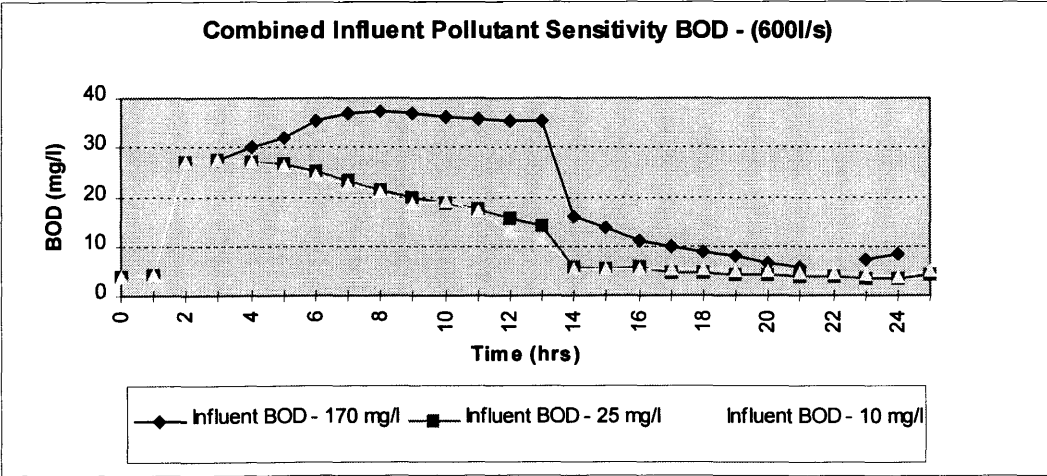


Fig. 15.19 Dilution Analysis at 600l/s - WTP Effluent (BOD)



Figures 15.18 and 15.19 show a very similar trend with respect to effluent quality as was obtained when the concentrations were applied at a flow rate of 1000l/s (figs.15.14 and 15.15). The maximum difference between the extreme influent TSS scenarios (205mg/l and 30mg/l) was 17%. The maximum difference between the extreme influent BOD scenarios (170mg/l and 10mg/l) was 190%. With respect to the analysis using an influent of 500l/s (figs 15.20 and 15.21) it can be seen that there has been a slight reduction in relative sensitivity with respect to varying influent TSS concentrations. The maximum difference between the extreme scenarios has been reduced to 12.5%. It can also be noted that at this lower flow rate substantially better effluent quality is produced. This is a consequence of the decreased surface loading rate providing a greater opportunity for the particles to settle.

Fig. 15.20 Dilution Analysis at 500l/s - WTP Effluent (TSS)

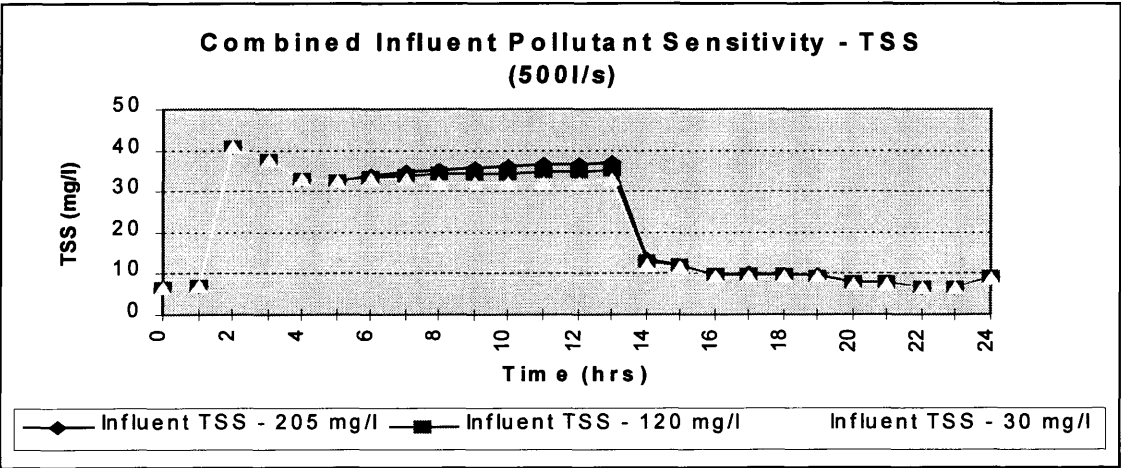


Fig. 15.21 Dilution Analysis at 500l/s - WTP Effluent (BOD)

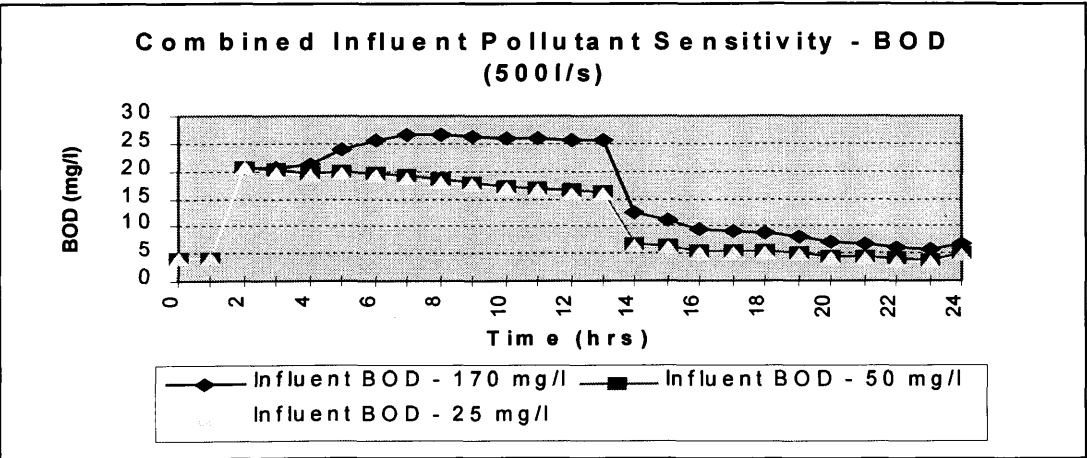


Figure 15.21 also shows a definite improvement with respect to BOD effluent concentrations when flows between 600 and 1000l/s were applied. This is a consequence of the increased residence time with the reactor allowing more complete degradation. The graph also shows a significant reduction in relative sensitivity between the varying influent concentrations. By comparing figure 15.21 with figure 15.15 it can be seen that the maximum difference in effluent concentrations has been reduced from 190%, at hour 13 to 56%. Consequently, it is apparent that even if the developed hypothetical profile was far removed from reality it would not result in significant errors at this lower flow rate. The trend of decreasing sensitivity is continued with decreasing flow rate.

Fig. 15.22 Dilution Analysis at 400l/s - WTP Effluent (TSS)

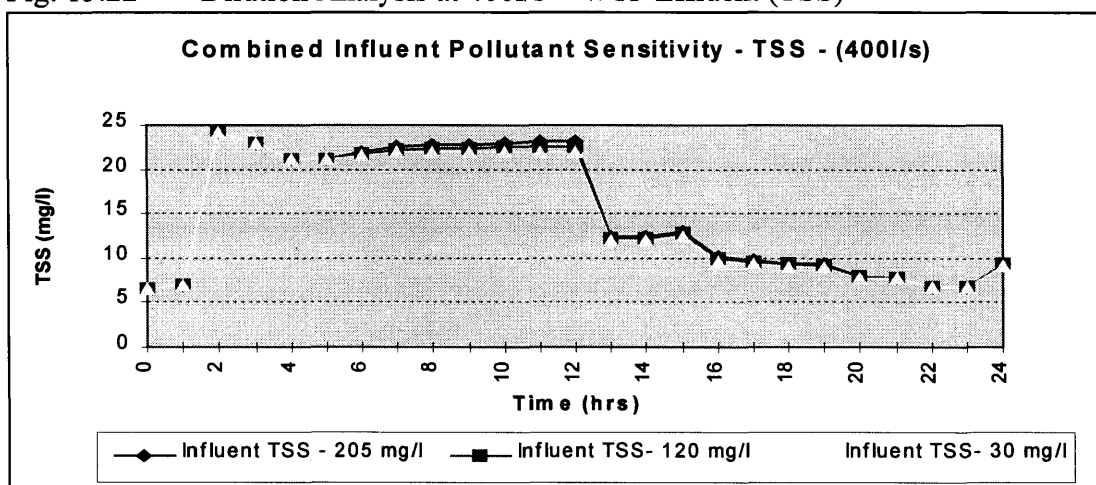


Fig. 15.23 Dilution Analysis at 400l/s - WTP Effluent (BOD)

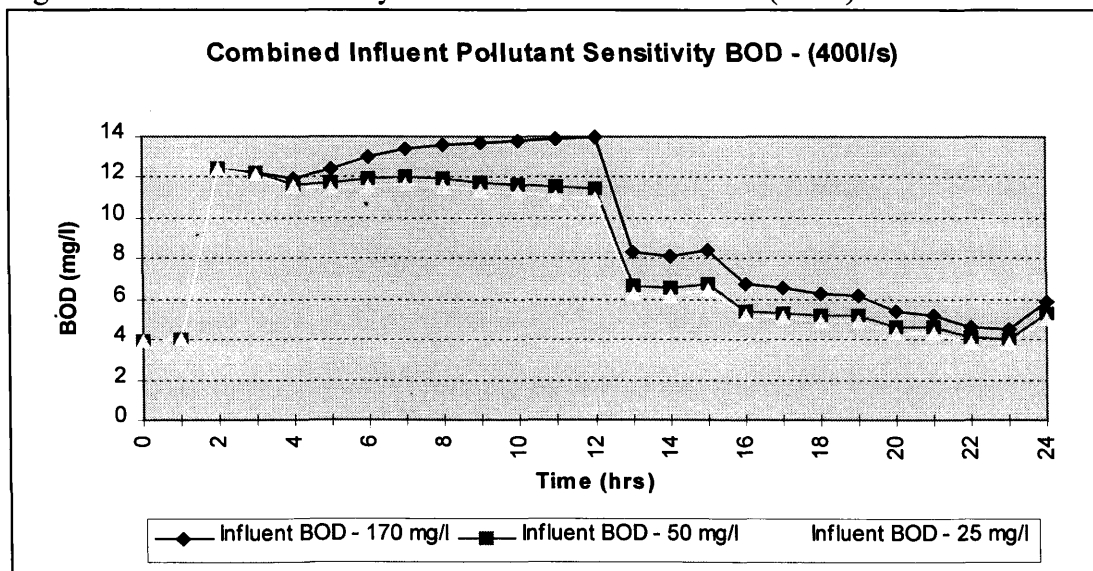


Figure 15.23 shows a maximum difference in effluent quality of 22% for the extreme influent BOD concentrations. Fig 15.22 shows a maximum difference of only 6%.

15.5 Conclusions for Sensitivity Test No.3

The results from the WTP sensitivity analysis have shown that only BOD was sensitive to varying influent concentrations. As expected the greatest sensitivity occurred at the maximum influent flow rate. However, even at this flow rate the sensitivity was not significant (influent concentrations of 10 and 25 mg/l produced an effluent difference of only 5 mg/l). The storm tanks however showed greater sensitivity, and dictated that significant errors could result if the hypothetical profile was poorly defined. These results therefore highlighted the necessity of discarding the sewer flow quality model and developing a hypothetical profile which would provide as accurate as possible sewer flow quality data.

15.6 Development of Hypothetical Influent Profile

TSS

The collected data at Sleepless Inch suggested that for an influent flow rate of 1000l/s the associated TSS concentration would be in the order of 120mg/l. It was shown from the sensitivity testing that if the actual concentrations varied from this defined value no significant errors would result. Consequently, a fixed TSS influent concentration of 120mg/l was defined for flow rates of 1000l/s.

BOD

The collected data at Sleepless Inch has shown that this determinand can be significantly diluted by high flows. Consequently, a standard BOD influent pollutant concentration of 25mg/l was chosen for flow rates generating 1000l/s. This value was taken from the data collection exercise. The sensitivity testing showed variability in effluent concentrations could be expected if this concentration was poorly defined. However, the associated error would require to be accepted as an unavoidable limitation. The sensitivity of the model to these

variations significantly decreased with decreasing flow rate. Consequently, the significance of defining inaccurate concentrations is decreased as the influent flow rate reduces to dry weather flow. This therefore resulted in an influent pollutant profile of 120:25 mg/l (TSS:BOD) for flow rates generating 1000l/s. The average DWF characteristics, as previously discussed in section 15.4 were defined as follows:- 205:170 mg/l (TSS:BOD) for flow rates of 300l/s. This therefore provided two points from which the hypothetical profile could be interpolated:-

1000l/s	120:25 mg/l (TSS:BOD)
300l/s	205:170 mg/l (TSS:BOD)

By interpolating between these two values the hypothetical influent pollutant profile was obtained. The profile is shown in table 15.11.

Table 15.11 Hypothetical Influent Pollutant Profile -TSS and BOD

Flow (l/s)	TSS (mg/l)	BOD (mg/l)
300	205	170
400	180	130
500	160	115
600	150	100
700	140	75
800	140	50
900	125	25
1000	120	25

The same procedure was adopted for the definition of the hypothetical ammonia profiles. Interpolation was carried out between the diluted concentration of 4mg/l which was observed to result for flows of between 900 and 1000l/s and the average dry weather concentration of 17 mg/l corresponding to a dry weather flow of 300l/s.

The interpolated hypothetical ammonia profile is shown in table 15.12.

Table 15.12 Hypothetical Influent Pollutant Profile - Ammonia

Flow (l/s)	Ammn (mg/l)
300	17
400	16
500	14
600	10
700	7
800	6.25
900	4
1000	4

Table 15.12 was considered to be a realistic estimate of influent ammonia concentrations for the various flow rates. No ammonia sensitivity testing was carried out as the WTP at Sleepless Inch does not nitrify. Consequently, the ammonia passes through the plant unmodified. The developed hypothetical WTP influent profile for the three determinands are shown in table 15.13:-

Table 15.13 Hypothetical Influent Pollutant Profiles - TSS, BOD and Ammonia

Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)
300	205	170	17
400	180	130	16
500	160	115	14
600	150	100	10
700	140	75	7
800	140	50	6.25
900	125	20	4
1000	125	20	4

Although these values are hypothetical they are based upon interpolation between values obtained from data collection and are justified by the conclusions obtained from sensitivity tests. It is accepted however that the utilisation of assumed pollutant concentrations is a limited approach to complex modelling. Nevertheless, it requires to be noted that this problem is not restricted solely to this research project but is a common problem related to all studies which require sewer flow quality data. This is emphasised by **Schutz (1998)** and **Guderain et al, (1997)** who both required to utilise 'assumed' pollutant concentrations for their analysis.

15.7 Development of Hypothetical CSO Spill Pollutant Data

South Inch Pumping Station Combined Sewer Overflow

Data collection at the South Inch overflow provided three suitable events from which to analyse storm flow quality. The principle adopted was to utilise these events to develop as accurate as possible 'flush component' pollutant concentrations and to utilise the data collected at the WTP to develop the hypothetical concentrations for the "diluted component" of the sewer flow quality.

The reason influent WTP data required to be used for the development of the "diluted component" data was because insufficient data were collected at the CSOs. This was because the events which were collected were of short duration and thus did not provide suitable information.

As no major sewerage data connected to the network downstream of South Inch it was believed that WTP data would provide meaningful information with respect to the sewer flow quality (diluted component) at the South Inch location. The events which were utilised in the development of the flush profiles were those occurring on the 17/5/95, 31/5/95 and 17/10/95. The characteristics of these events are shown in table 15.14.

Table 15.14 Rainfall Characteristics

Date	Rain Gauge -Murray Royal (Peak & Vol.)	Rain Gauge - Burghmuir (Peak & Vol.)	Rain Gauge - Perth Grammar (Peak & Vol.)
17/5/95	3 mm/hr 1 mm	14 mm/hr 5.6 mm	18 mm/hr 4.2 mm
31/5/95	24 mm/hr 7.4 mm	45 mm/hr 5.8 mm	7 mm/hr 2.2 mm
17/10/95	12 mm/hr 3.2 mm	6.4 mm/hr 0.4 mm	6.5 mm/hr 1.4 mm

The resulting sewer flow and quality data with respect to the flush components are shown overleaf:-

Event 17/5 (ADWP 10 days)

Duration of storm flows: No Flow data (On-site observations - low flows)

Peak TSS:- 650 mg/l

Peak BOD:- 250 mg/l

Event 31/5 (ADWP 3 days)

Duration of storm flows: No flow data (On-site observations:- significant flows)

Peak TSS 625 mg/l

Peak BOD 95 mg/l

Event 17/10 (ADWP 1 day)

Duration of storm flows: 1hr (on site observations:- significant flows)

Peak Flow 880l/s

Peak TSS:- 350mg/l

Peak BOD:- 180 mg/l

The flow and quality characteristics for the three events are summarised below:-

Table 15.15 South Inch Flow and Quality Characteristics

	Event 17/5/95 (ADWP 10dys)	Event 31/5/95 (ADWP 3 day)	Event 17/10/95 (ADWP 1 day)
Peak Flow	no data (weak event)	No data (significant event)	880l/s
Peak TSS	650 mg/l	625 mg/l	350 mg/l
Peak BOD	250 mg/l	95 mg/l	180 mg/l

TSS

A significant difference can be observed with respect to the maximum TSS concentrations which occurred on the 17/10/95 and the flush concentrations which occurred on the 17/5 and the 31/5/95. It can be seen that the 17/10 event although producing a high flow rate, did not give rise to a substantial flush. The reason for this is believed to be due to the ADWP, which for that event, was only one day. For the 17/5 event which had a larger ADWP, it was noted that the low flow rates produced peak TSS concentrations of around 650mg/l. As insufficient data were available to explicitly relate possible flush concentrations with flow rate and ADWP a simplistic conservative approach required to be adopted.

In this approach it was assumed that the maximum flush concentration could result each time an event occurred, regardless of flow rate. Consequently, the loads being spilled to the receiving watercourse would be limited solely by the hydraulics of the system i.e. larger events would cause greater loads to be discharged to the receiving watercourse in comparison with smaller events because the larger events would give rise to greater spill volumes. Although this would result in an over prediction of pollutant loads being spilled to the receiving watercourse for low intensity events it would ensure that the analysis carried out was conservative. In order to define this maximum flush concentration, certain assumptions required to be made with respect to the ADWP and resulting flush concentrations. The above data suggested that due to the sensitivity of the flush with respect to ADWP it would be possible for concentrations greater than 650mg/l to occur if the ADWP were larger than 10 days. Consequently, to limit possible flush concentrations an upper bound assumption was made that the maximum TSS flush concentration which could result would be 850mg/l.

BOD

BOD flush concentrations, on the other hand showed a relative insensitivity to ADWP and consequently it was hypothesised that events with ADWP of greater than 10 days would not significantly increase the peak BOD concentration. However, to ensure conservatism a peak value of 450mg/l was chosen as the upper bound BOD flush concentration. Consequently, this resulted in hypothetical ‘‘flush’’ data of **850:450mg/l** TSS:BOD being derived for the South Inch overflow.

The qualitative data for the ‘‘diluted component’’ of the pollutograph were taken from the WTP. As it was known that the length of sewer running from the South Inch to the WTP is affected by infiltration an arbitrary pollutant concentration correction of +10% was utilised to take this into account. This is shown in table 15.16

Table 15.16 Development of South Inch Diluted Component Hypothetical Data

Hypothetical WTP Pollutant Data – (Diluted Component)				Hypothetical South Inch Pollutant Data (Diluted Component)			
Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)	Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)
300	205	170	17	300	225	187	18.7
400	180	130	16	400	198	143	17.6
500	160	115	14	500	176	126	15.4
600	150	100	10	600	165	110	11
700	140	75	7	700	154	82	7.7
800	140	50	6.25	800	154	55	7
900	125	20	4	900	137	22	4.4
1000	125	20	4	1000	137	22	4.4

Friarton Pumping Station

The Friarton Pumping station is located just down-stream approximately 200m of the South Inch overflow it was deemed feasible to apply the same hypothetical data at this site.

Willowgate Pumping Station

Only two events were suitable from which to develop maximum “flush component” concentrations at this location (17/5/95 and 31/5/95). The characteristics for which were shown in table 15.14. The corresponding sewer flow and quality data are shown below:-

Event 17/5

(ADWP 10 days - On-site observations - reasonable storm flows)

Duration of storm flows: 1hr
Peak Flow: 120l/s
Peak BOD 325 mg/l
Peak TSS 580mg/l

Event 31/5 (ADWP 3 days)

Duration of storm flows: 1hr
Peak Flow 180l/s
Peak BOD 300 mg/l
Peak TSS 980mg/l

From analysis of the data the summary table 15.17 was produced:-

Table 15.17 Bridgend Flow and Quality Characteristics

	Event 17/5/95 (ADWP 10dys)	Event 31/5/95 (ADWP 3 day)
Peak Flow	120l/s	180l/s
Peak TSS	580 mg/l	980mg/l
Peak BOD	325 mg/l	300mg/l

It can be deduced from table 15.17 that a complex relationship exists between the ADWP, flow rate and the resulting flush concentrations. This is particularly evident at the Willowgate PS on the 31/5 as the high flow rate gave rise to a substantial release of granular sediment (as the ADWP is small). The 17/5 event had a higher ADWP, yet a lower flow rate, and gave rise to a lower TSS flush but a similar BOD flush. As previously discussed this complex relationship could not be taken into consideration due to the limited data which were available. It was therefore assumed that the maximum flush concentration which could occur at this location would be 1000 mg/l TSS and 450 mg/l BOD. These factors would be applied to all discharges regardless of the hydrograph. This provided ‘‘flush component’’ data for the Willowgate Pumping Station of **1000:450** mg/l (TSS:BOD). As no suitable collected quality data were available to develop pollutant characteristics for the ‘‘diluted component’’ of the combined flows, the characteristics defined at the South Inch pumping station were utilised, albeit with an arbitrary +15% correction factor to account for routing and infiltration within the system. This is demonstrated in table 15.18:-

Table 15. 18 ‘‘Diluted Component’’ Hypothetical Data

South Inch Weighting Factors				Willowgate Weighting Factors			
Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)	Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)
300	225	187	18.7	300	258	215	21.5
400	198	143	17.6	400	227	164	20.2
500	176	126	15.4	500	202	144	17.7
600	165	110	11	600	189	126	12.65
700	154	82	7.7	700	177	94	8.8
800	154	55	7	800	177	63	8
900	137	22	4.4	900	157	25	5
1000	137	22	4.4	1000	157	25	5

Tables 15.19 and 15.20 show the derived “flush” and “diluted component” concentrations for the overflows at the South Inch, Friarton and Willowgate pumping stations:-

Table 15.19 Flush Component Characteristics

	South Inch	Friarton	Willowgate
TSS (mg/l)	850	850	1000
BOD (mg/l)	450	450	450

Table 15.20 Diluted Component Characteristics

South Inch Pumping Station				Friarton Pumping Station			Willowgate Pumping Station		
Flow (l/s)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)	TSS (mg/l)	BOD (mg/l)	Ammn (mg/l)
300	225	187	18.7	225	187	18.7	258	215	21.5
400	198	143	17.5	198	143	17.5	227	164	20.2
500	176	126	15.5	176	126	15.5	202	144	17.7
600	165	110	11	165	110	11	189	126	12.65
700	154	82	7.7	154	82	7.7	177	94	8.8
800	154	55	7	154	55	7	177	63	8
900	137	22	4.4	137	22	4.4	157	25	5
1000	137	22	4.4	137	22	4.4	157	25	5

The development of these data allowed the research to continue.

15.8 Summary and Conclusions

The aim of the sensitivity work carried out in this chapter was primarily to help develop hypothetical quality data for use in the research project. The initial sensitivity test showed that an interval of two hours would be acceptable for WTP input data. However, it was decided to utilise an input interval of only one hour as this would allow a more accurate “flush” profile to be defined. Test No.2 showed that a standard WTP “flush” of 800:650mg/l (TSS:BOD) could be applied for all ranges of in flow. This was a significant finding as it greatly aided the WTP influent file preparation. Sensitivity test No. 3 was concerned with the ascertaining the magnitude of the error which could result if the “real” WTP influent data was different from the developed “hypothetical” profile. It was noted that the storm tanks showed the greatest sensitivity to variations in influent concentrations. It was therefore concluded that if the sewer flow quality model

was utilised to provide WTP input data, erroneous storm tank effluent data could result. With respect to the WTP effluent it was observed that BOD was a more sensitive parameter than TSS. This was believed to be a consequence of the more complex processes required for its removal. Nevertheless, it was observed that if an educated attempt was made at defining the influent concentration the potential error could be significantly reduced. Using the information from these tests the hypothetical WTP quality influent profile was developed.

Flush characteristics for the three CSOs in the sewerage system were conservatively based on the data provided from the sewer flow quality data collection exercise. This was because insufficient data were available to attempt to regress the flush concentrations against rainfall, flow and the antecedent dry weather period. If sufficient data were available this would have been the adopted approach. The diluted component of the combined flows at the CSOs were derived from the hypothetical WTP data albeit with arbitrary correction factors applied to account for routing and infiltration. It was believed that the hypothetical data derived in this chapter were the most appropriate data which could be used in the research.

Chapter Sixteen

Method Development

16.1 Introduction

As discussed in chapter one, aims of this project were to substantiate and quantify the total emission problem by means of detailed modelling analysis and to produce a method which could be used by engineers to resolve total emission problems within their own particular drainage catchment. This chapter provides a detailed explanation of the novel method which was produced to carry out the analysis. The method was referred to as the Total Emission Analysis Period (TEAP) method.

16.2 Analysis Period

It was established in chapter five that if useful information was to be provided with respect to total emissions (in terms of acute pollution) the time scale for the analysis would require to be much shorter than one year. As the nature of acute pollution is one of short term problems within a receiving water course, typically during and immediately after an event (**Harremoes, 1989**), it was initially believed that the analysis could be carried out simply on an event to event basis. However, with reference to chapter two, which discusses WTP performance with respect to combined loads, it is apparent that misleading results can also be obtained if the chosen analysis period is too short i.e. if the drainage of a storage volume caused disruption at a plant for a period of two days, where, for the same event, a relatively larger storage volume disrupted the plant for a period of six days, a true comparison of total emissions with respect to varying storage volume could only be ascertained if the analysis were carried out over the longer period. This emphasised the need for a reference analysis period from which the WTP loads, with respect to all storage volumes being analysed, could be compared.

With reference to the above example it was apparent that this reference period would require to be based on the disruptive period which the largest storage volume (which is likely to be used in the catchment) would cause, as this volume would give rise to the greatest period of disruption at the plant (for the same given rainfall profile).

Furthermore, the greater the period of disruption occurring at the plant, the greater the potential for additional rainfall occurring within this period. If such rainfall were to occur whilst the plant was still in a disruptive state, even further disruption could result, thus requiring the analysis period to be extended further still. Utilising the UPM manual (FWR, 1994) to demonstrate these points, it can be seen that three Time Series Rainfall Events (TSR) events, with characteristics as shown below, were used in a total emission assessment:-

Table 16.1 TSR Characteristics and Total Emission Analysis Period

Event No.	Volume (mm)	Mean Intensity (mm/hr)	Peak Intensity (mm/hr)	Duration (hrs)	Total Emission Analysis Period (hrs)
3	26.3	4.38	6.8	6	10
4	20.7	1.88	5	11	13
12	14.4	3.6	6	4	8

From table 16.1 it can be seen that the simulations have been carried out for a period only slightly longer than the duration of the event. Figs. 16.1-16.3 show that for the scenario where storage has been utilised, the analysis period has not even been sufficient to allow the flows to return to dry weather. It can also be deduced from the figures that the plant is still in a disruptive state at the end of the analysis period. i.e. from fig 16.2 it can be calculated that effluent BOD and Ammonia concentrations under dry weather conditions are approximately 11mg/l and 1.3mg/l, respectively, whereas at the end of the simulation the effluent concentrations are still higher, 18.5mg/l and 1.85mg/l, respectively.

Fig. 16.1 WTP Emissions for TSR Event 3 (source - FWR, 1994)

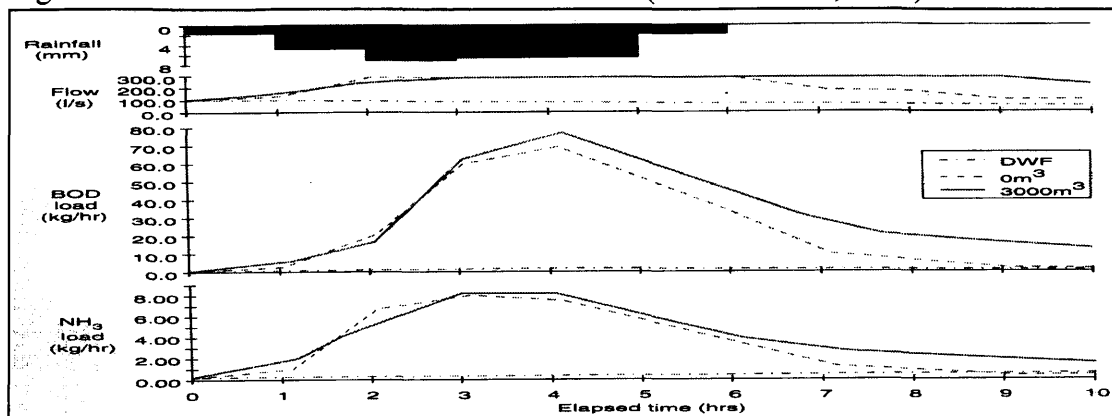


Fig. 16.2 WTP Emissions for TSR Event 4 (source - FWR 1994)

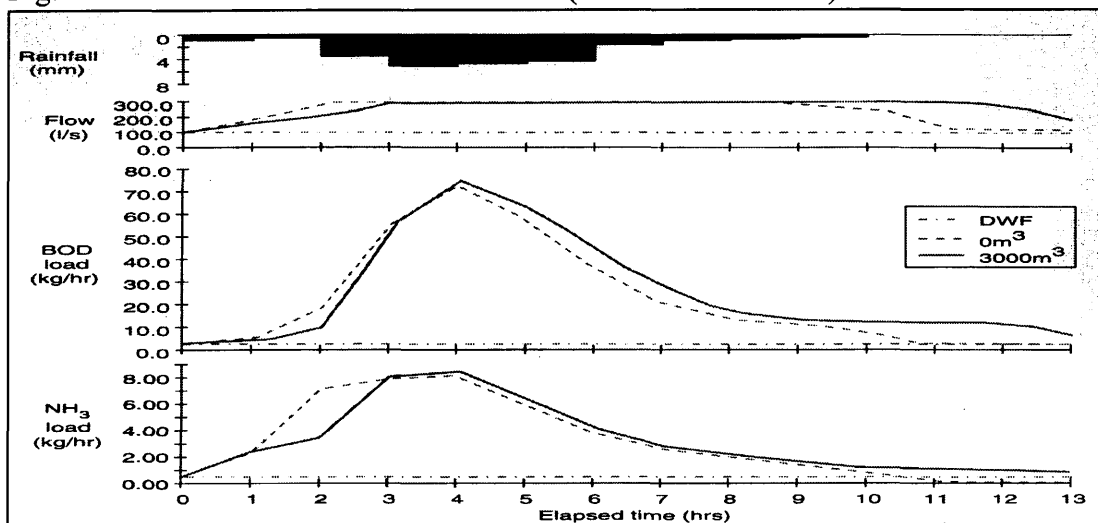
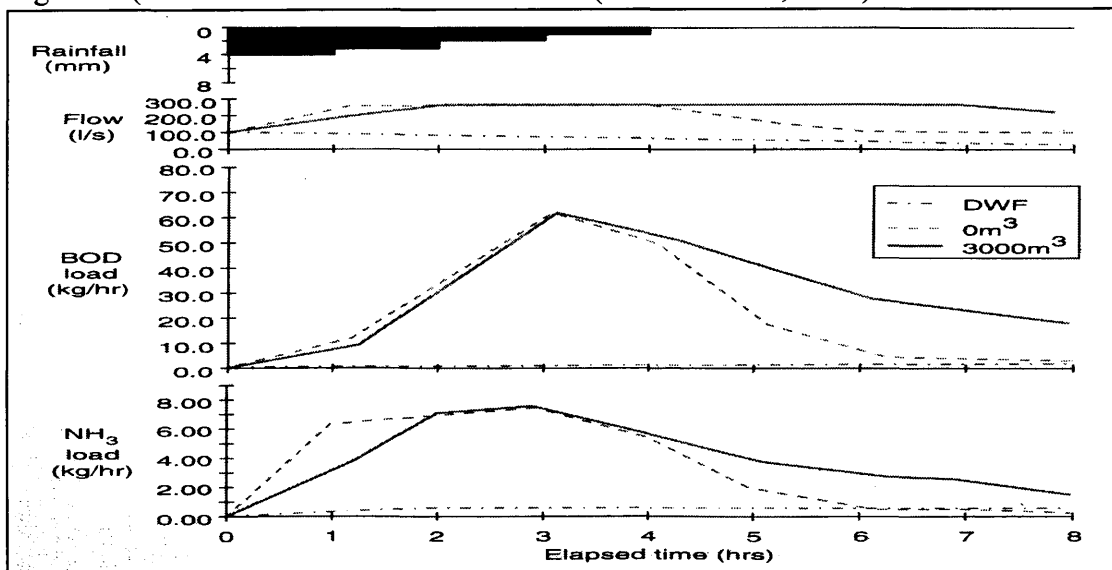


Fig 16.3 (WTP Emissions for TSR Event 12 (source - FWR, 1994)



It is therefore demonstrated that the analysis period as defined above would not be sufficient to allow a representative comparison of WTP emissions with respect to varying storage volumes. This would be particularly apparent if larger storage

volumes were utilised which produce even longer drain times. Furthermore, as this analysis has been concerned with discrete event analysis the possibility of subsequent rainfall events occurring whilst the plant is still in a disruptive state is ignored. If an additional rainfall event were to occur before the plant had restored itself to normal functioning the effluent quality could be expected to be worse than it would have been if the plant were operating at a steady state. This could have implications with respect to discharge consents.

16.3 Continuous versus Discrete Simulation

Long disruptive periods at a plant increase the potential for additional rainfall occurring within the disruptive period. The combination of events can serve to exacerbate the disruption experienced at the plant, and thus the effluent quality. Only through continuous simulation can this be evaluated properly. This is because discrete event analysis requires pre-defined initial conditions for the WTP at the start of each rainfall. Unless continuous simulation is utilised, the pre-defined initial state of the WTP could be far removed from reality, giving rise to erroneous results (the biological unit processes are not operating at steady state at the onset of the event, the storm tanks are not empty etc.). Consequently, it is a possibility that discrete event analysis could show that receiving water quality standards are passed for each event under consideration whereas the same events, analysed using continuous simulation, may show the receiving water quality standards to be breached. Consequently, continuous simulation would provide the most accurate results with respect to these problems. Continuous simulation is also a necessity for CSO discharges, as the storage tanks may be anywhere from empty to completely full at the onset of the rainfall event(s).

16.4 Summary

- The analysis of total emissions with respect to acute pollution requires to be carried out over a time-scale much shorter than one year, however, it is not possible to analyse the emissions simply on an event to event basis as discrete simulation can give rise to misleading and erroneous results.

- The analysis period should be of a 'reference' duration from which the emission from all storage volumes can be compared.
- Since draining of storage tanks can cause significant disruption to the WTP, even after the dry weather flows are re-established, the reference analysis period requires to be sufficiently long to prevent various storage volumes appearing to produce similar emissions.
- Due to the possibility of additional rainfall events occurring whilst the plant is in a disruptive state, causing further disruption to the plant and poorer WTP effluent, the reference analysis period requires to be of a suitable duration to take this into consideration.

It was thus a primary aim of the project to produce a method which could calculate a suitable analysis period, meeting the following two criteria;

1. that it would not be too short that unrepresentative and misleading information would result.
2. that it would not be so long that the information provided would be of greater relevance to chronic rather than pollution problems and/or require too much computer time.

A method was therefore produced to meet these criteria, and was referred to as the Total Emission Analysis Period (TEAP) method.

16.5 Maximum Storage Volume

With reference to the previous discussion it was proposed that the reference TEAP should be calculated based upon the recovery/disruptive period which the most likely maximum storage volume (which could be used in a catchment) would cause at the WTP, for a given range of rainfall events. This maximum volume would be site specific and dependent upon a number of factors such as the spill frequency of the CSOs, the sensitivity of the receiving watercourse and,

for example, the practical limitation of locating storage within the catchment. It is important to note that the maximum volume chosen should not be a value chosen arbitrarily, but one based on the requirements of the catchment as defined by the aforementioned site specific criteria.

16.6 Rainfall Profiles

In order to reduce the simulation times it is proposed that only the worst case conditions in a typical year are analysed. This follows from the UPM acute pollution discharge standards which provide allowable return periods for low dissolved oxygen and high ammonia concentrations. Analysis by **Crabtree et al, (1993)** has shown that the one year RP is most likely to be critical under the majority of situations. Thus, if the one year return period thresholds are met, the three and one month thresholds should also be met (**FWR, 1994**). In terms of the proposed total emission analysis this greatly reduces the amount of simulations which are required to ascertain whether a range of storage volumes would provide statistical compliance with the discharge constraints. The reason for this is that the one year return period criteria requires to be breached only twice in a typical year for the standards to be failed. Consequently, only the worst case scenarios in the year require to be analysed. Tables 16.2 and 16.3 show the Fundamental UPM Acute Pollution Standards:-

Table 16.2 Fundamental UPM Acute Pollution Standards (DO)

Return Period	DO Concentrations (mg/l)		
	1 hr	6 hrs	24 hrs
1 month	4.0	5.0	5.5
3 months	3.5	4.5	5.0
1 year	3.0	4.0	4.5

Table 16.3 Fundamental UPM Acute Pollution Standards (Un-ionised Ammonia)

Return Period	Unionised Ammonia Concentrations (mg/l)		
	1 hr	6 hrs	24 hrs
1 month	0.150	0.075	0.030
3 months	0.225	0.125	0.050
1 year	0.250	0.150	0.065

16.61 Worst Case Scenarios

The periods which would undoubtedly require to be analysed would therefore be the wettest months over the summer and winter periods. Winter is considered to be a critical period as the rainfall in this season is frequent and of long intensity. Such events would cause the greatest disruption at the WTP and would result in the greatest loads being discharged to the receiving watercourse (although the watercourses should have greatest assimilation capacity).

Summer is also critical as the prolonged dry spells reduce the base flows within the receiving watercourse and lower the assimilative capacity. The nature of the weather in this season does however minimise the possibility of prolonged periods of poor WTP performance and thus it requires to be noted that the adverse factor may not be the affect of storage upon WTP performance, but simply the low assimilative capacity within the receiving watercourse.

Using only the worst case scenarios it could therefore be shown that the standards are met or failed without the need for extensive simulation. This has important implications with respect to the time requirements for the analysis of ten or more years of historical data (which are required to produce statistically valid predictions of pollutant concentrations within a watercourse) analysis times will be significantly reduced.

16.7 Definition of Wettest Periods

In order to define the wettest periods it recommended that either historical rainfall profiles or statistically generated profiles are utilised. As previously discussed in section 16.61 above, it is suggested that initially the wettest months in summer and winter (defined by volume) are utilised and from these profiles the reference TEAP can be calculated. In order to define the required TEAP a 'rain-day' method has been suggested. In this method a table is produced showing the days from the wettest month in which rain fell. Table 16.4 provides an example table.

Table 16.4 Rain-Day Table

Date	Duration (hrs)	Depth (mm)	Mean Int. (mm/hr)	Max. Int. (mm/hr)	Start Time (hrs)
1	13	3	0.2	0.4	04:00
2					
3					
4					
5					
6	20	32.4	1.6	5.3	15:00
7	14	14.8	1.1	1.8	14:00
8	5	11.5	2.3	2.4	12:00
8	4	2.3	0.6	0.9	19:00
9	10	9.8	1	2.9	4:00
10					
11					
12	4	1.5	0.4	0.6	18:00
13	3	1.2	0.4	0.6	19:00
14					
15	8	7.2	0.9	2.6	18:00
16	3	1.3	0.4	0.7	07:00
17					
18	1	2.3	2.3	2.3	4:00
19					
20	4	3.1	0.8	2	8:00
20	9	5.4	0.6	1.8	14:00
20	3	1.3	0.4	0.8	24:00
21					
22					
23	4	1	0.3	0.4	22:00
24	5	2.5	0.5	1.2	03:00
24	8	5.3	0.7	2	14:00
24	3	1.2	0.4	0.5	24:00
25					
26	2	1	0.5	0.5	04:00
27					
28					
29					
30	4	5.2	1.3	2.1	6:00
30	4	3.7	0.9	1.6	18:00

As previously discussed, it is only necessary to analyse the worst case period within this wettest month as this period would cause the worst disruption at the plant. It can be seen from table 16.4 that in this particular example the worst period begins on the 6th day. It is proposed that the following equation is then used along with the 'rain-day' table to determine the required duration of the analysis period:-

$$T = D + R \quad \text{eqn. 16.1}$$

where

- T = Total Emission Analysis Period (hrs)
D = Duration of Combined Loading resulting from initial event (hrs)
R = WTP Recovery Period (hrs)

Utilising the above equation a check would then be made to determine whether the plant would restore itself to normal functioning prior to the onset of the second rainfall event. If it is found that additional rainfall occurs within the recovery period, the analysis period requires to be extended to account for the additional disruption which the subsequent event(s) cause.

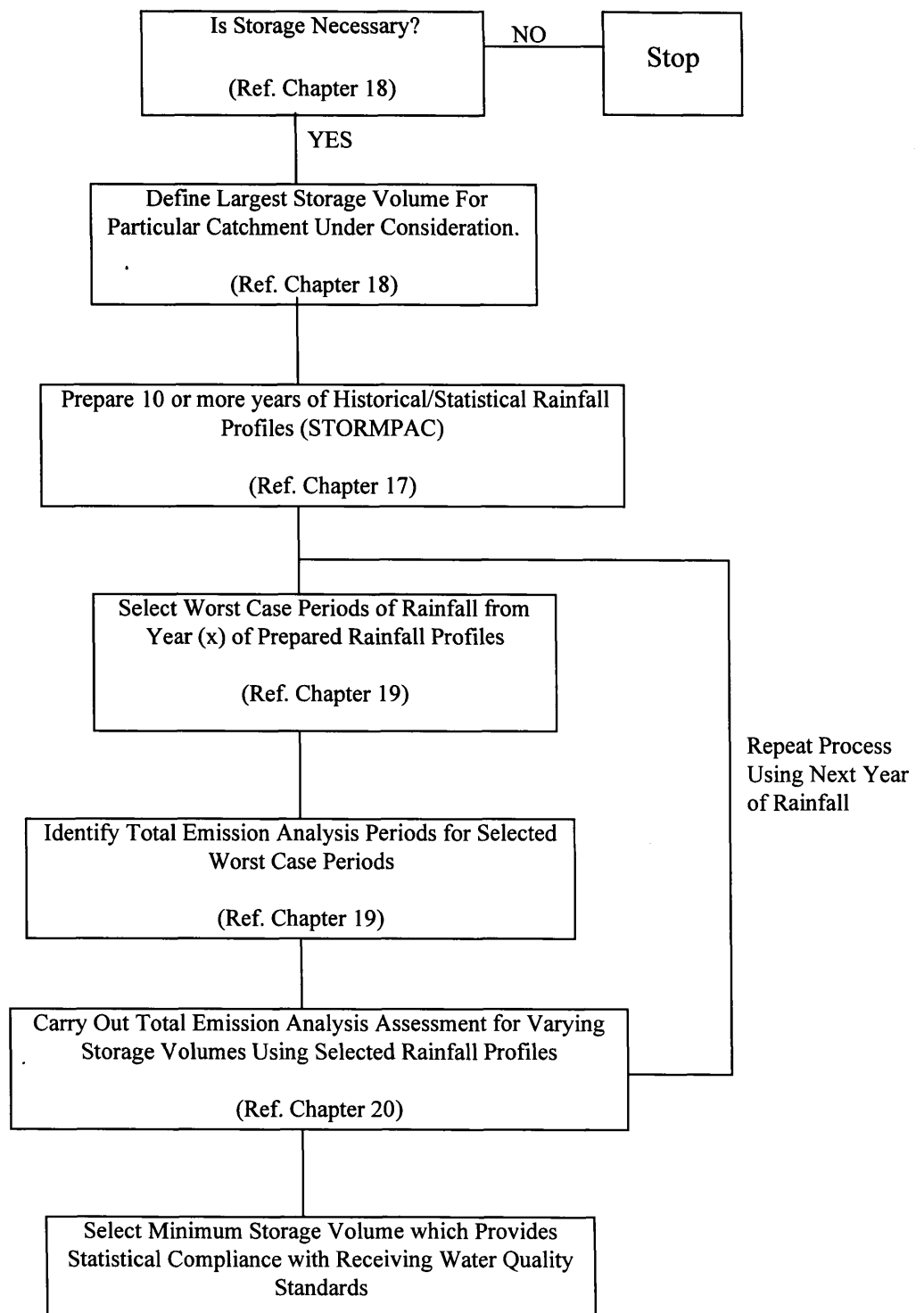
The expected duration of the disruptive periods can be obtained from sensitivity analysis of WTP performance with respect to varying influent characteristics. i.e. the rainfall events will result in a combined loading period of 'x' hrs, which causes disruption at the treatment plant for 'y' hrs.

Once the TEAP has been defined the actual analysis can then be carried out.

This same procedure requires to be carried out for the other worst period(s) in the year, repeating the entire process for the ten years of rainfall data. This analysis could then show which storage volumes (if any) cause a 'statistical' breach of receiving water quality standards.

The method is shown diagrammatically in Figure 16.4 overleaf:-

Figure 16.4 TEAP Method Overview - Flow Chart



The subsequent chapters provide more information and worked examples for each of the above steps in the method.

16.8 Method Summary

- Total Emission Analysis requires to be carried out over a reference duration which is long enough to provide representative results for each storage volume under consideration.
- The reference duration was referred to as the Total Emission Analysis Period (TEAP).
- The TEAP requires to be based on the disruptive period which the largest storage volume used in the analysis causes.
- The largest storage volume should not be chosen arbitrarily but be based on site specific criteria such as the spill frequency of the CSOs, the sensitivity of the receiving watercourse and the practical limitation of locating storage within the catchment.
- At least ten years of historical rainfall data should be used for the analysis.
- The drainage catchment solution must ensure that the 1 year UPM acute pollution standards are met. This is because work has shown that these standards are critical.
- Only the worst case rainfall periods which occur (defined by volume) require to be analysed i.e. not every rainfall event requires to be analysed.
- A "rain day" table can be used to analyse the periods of rainfall which should be analysed.

Chapter Seventeen

Application of Proposed Method to Perth - Rainfall Generation

17.1 Introduction

As introduced in chapter sixteen, total emissions should be analysed using ten or more years of rainfall data to determine whether the system statistically meets, or fails, the discharge consents. In order to provide long term rainfall profiles for use within the Perth study the STORMPAC rainfall generation model was utilised.

17.2 STORMPAC

STORMPAC was developed as part of the Urban Pollution Management research programme to provide long term statistical rainfall profiles for any location in the U.K. (Threlfall and Cowpertwait, 1996). The package utilises a version of the Newmann Scott Cluster Poisson process and five estimated rainfall parameters, as listed below, to generate the rainfall:-

- 1) The mean waiting time between the beginning of storms
- 2) The mean number of rain cells per storm
- 3) The mean duration of each rain cell
- 4) The mean intensity of each rain cell
- 5) The mean waiting time for each rain cell after the beginning of the storm.

The above parameters were estimated from rainfall statistics taken from historical records at 112 sites randomly scattered throughout the UK. and regressed using rainfall characteristics which are known to influence rainfall. these characteristics are location, altitude, average annual rainfall and distance from coast. The

accuracy of the SRG model has been tested in practical applications (**Threlfall and Cowpertwait, 1996**) and was found to be suitable for investigations requiring storms with return periods of less than 10 years.

17.3 Perth

The four site-specific rainfall characteristics as discussed above were utilised to develop the long term rainfall profiles for the Perth catchment. These data are listed in table 17.1.

Table 17.1 Perth Rainfall Generation Characteristics

Location	Perth (Northing 7230, Easting 3130)
Altitude	76m
Average Annual Rainfall	750mm
Distance from Coast	20 miles

Daily average rainfall information were also input into the model to aid the regionalisation process. Table 17.2, presented by **Wotherspoon et al, (1996)** compares the monthly rainfall volume as generated by STORMPAC with published monthly volumes for the Perth area. It can be seen that STORMPAC has provided very good results.

Table 17.2 Monthly Average Rainfall (MAR) - Published and STORMPAC

	MAR (mm) Published Data	MAR (mm) STORMPAC
JANUARY	86.1	96.0
FEBRUARY	52.4	49.1
MARCH	65.6	60.3
APRIL	41.6	44.2
MAY	47.4	45.7
JUNE	57.5	63.1
JULY	58.1	50.3
AUGUST	63.6	69.6
SEPTEMBER	67.4	69.5
OCTOBER	74.2	69.2
NOVEMBER	66.5	64.5
DECEMBER	73.9	77.6
ANNUAL TOTAL	754.3	759.1

A further check was made to ascertain the integrity of the distribution of the events within the synthetic series. This was carried out by comparing the daily rainfall totals across a range of total depths.

Table 17.3 Daily Rainfall Totals – Published and STORMPAC

Daily Total Depth	<3 (mm)	$3 \leq x \leq 8$ (mm)	$0 \leq x \leq 15$ (mm)	>15 (mm)
Perth Data	78.6	15.9	2.9	2.6
STORMPAC	78.5	17	2.7	1.8

Table 17.3 shows that STORMPAC has over predicted the number of events falling in the 3 to 8mm range, whereas it under predicted the number of events falling in the >15mm range. In general, however STORMPAC has represented very adequately the historical rainfall profiles for the City of Perth. Consequently, these rainfall profiles were used for the Perth Total Emission Study.

Chapter Eighteen

Largest Possible Storage Volume Assessment for Perth

18.1 Introduction

Following the method as detailed in figure 16.4, the next stage in the process, subsequent to the generation of the rainfall profiles, was to determine the “largest possible” storage volume which should be used in the total emission analysis. As emphasised in chapter sixteen, this volume should not be chosen arbitrarily but should be based on characteristics and circumstances appropriate to the particular catchment under consideration. Such characteristics would be CSO spills (volume and frequency), and the assimilative capacity of the receiving watercourse. The “largest possible” storage volume should not be considered as the solution to a receiving water quality problem, but the storage volume from which the reference TEAP is calculated. The reference TEAP requires to be based on the largest storage volume used in the analysis to ensure that any comparison of total emissions with respect to varying storage volumes is meaningful (ref. 16.2). The actual volume which will resolve the receiving water quality problems will therefore be equal to or less than the defined “largest possible” storage volume. It can not however be greater than this volume. This chapter details how the ‘largest possible’ storage volume was defined for the Perth system.

18.2 General Procedure

The first stage in the process is to analyse whether or not the system requires storage. Notwithstanding the UPM guidelines, an initial storage assessment can be carried out using the Scottish Development Department (SDD) Method as this takes into consideration the dilution capacity provided by the receiving watercourse (SDD, 1977 and NRA, 1993). The SDD method states that CSO

storage would be necessary if the receiving watercourse provides a dilution of less than 8:1. This dilution is based upon the ratio of the low river flow to the dry weather flow in the sewerage system. The storage requirements are shown in table 18.1.

Table 18.1 SDD Storage Requirements

Dilution	Overflow Arrangement
8	Formula A
4	Formula A + storage of 40l/head
2	Formula A + storage of 80l/ head
1	Formula A + storage of 120l/head

The table shows that where the dilution in the receiving watercourse is 8:1 or greater, the overflows must pass the flows as calculated by Formula A. Formula A is the minimum flow rate which the sewerage system must pass before spill occurs.

Formula A

=

DWF + 1360P + 2E (l/day)

eqn. 18.1

where

DWF

=

Dry Weather Flow (l/day)

P

=

Population Served

E

=

Trade Effluent flow (l/day)

For a receiving watercourse which provides a dilution of 8:1 or greater it is deemed that CSO storage is not necessary. If storage is not necessary total emissions analysis would obviously not be required (as it is storage which causes the total emission problem).

For catchments where CSO storage is necessary it is suggested that the maximum storage volume be calculated via the SDD method with an applied +100% safety factor. The reason for the large safety factor being that the SDD method produces only an indication of the storage volume which requires to be utilised. This volume would require to be analysed with the ‘Fundamental Intermittent

Discharge Standards' or the 'Derived Intermittent Discharge Standards' (FWR, 1994) to ascertain if the volume is acceptable. It is therefore a possibility that the analysis may dictate a larger volume is required. As this report is concerned with calculating the maximum possible storage volume (to keep all analysis relative) a large factor of safety has been suggested. It is also essential that an evaluation of the performance of the overflows in the system is carried out in order to highlight the critical overflows in the system along with their expected spill ranges. This would highlight the need for improvements etc. Such an evaluation can be carried out quickly using the WRc SIMPOL (hydraulic) model (FWR, 1994) or using recognised techniques such as Method 1 analysis (Henderson and Dempsey, 1990).

18.3 CSO Storage and Flood Storage Protection

If a catchment requires a combination of storage to protect against flooding and to reduce CSO operation it is recommended that prior to the CSO spill analysis and the total emission analysis, the hydraulic model should be updated if necessary to account for the volume of storage which is likely to be used to protect the catchment against flooding. The reason being that such storage volumes prevent sewage being lost from the system and thus increase continuation flows to overflows and thus the possibility of greater spilled volumes. If for example the CSO spill assessment has been carried out neglecting the influence of the storage volume utilised to protect against flooding, inaccurate spill volumes can be obtained. The upgraded hydraulic model, which then accounts for the volume of storage required to protect the catchment against flooding, would be referred to as the base system (i.e. the effective no storage system) for the total emission analysis. This therefore allows a clear differentiation to be made between CSO storage and flooding storage. This is important since the two storage types give rise to different problems. The CSO storage can exacerbate WTP performance (thus giving rise to a potential total emission problem) due to the prolonging of combined influent, whereas the flood storage may not actually prolong the combined loads to the WTP but increase CSO emissions. If meaningful comparisons are to be made with respect to total

emissions in terms of varying storage volumes it is recommended that the flood storage volumes are removed from the analysis to prevent any confusion.

18.4 Perth Case Study

Following the guidelines laid down in section 18.2 an assessment was made of the required storage volume required for the Perth system. This was carried out following the SDD method of evaluation. The data used in the assessment are shown below:-

River Tay $Q_{95} = 44\text{m}^3/\text{s}$, Sewerage System PDWF = 250 l/s (at South Inch Overflow)

dilution ratio = $44,000\text{l/s} / 250\text{l/s} = \underline{\underline{176:1}}$

This ratio far surpasses the SDD storage criteria of 8:1 dilution. Consequently, *following the SDD guidelines* no CSO storage was necessary to protect the River Tay from pollutant discharges.

Due to the very high dilution capacity of the River Tay and the knowledge that no receiving water quality problems exists (and the knowledge that the WTP does not suffer from significant operational problems during combined loading) it was concluded that total emissions analysis would not be necessary. Even if storage were utilised in the Perth system, which served to increase the total emissions, an acute pollution problem would still not be expected to arise due to the high assimilative capacity of the receiving watercourse. This refers to economic criteria which dictates that even if total emissions were increased a problem would only arise if the receiving water quality standards were breached. As a receiving water quality problem is not likely in the River Tay no total emission analysis was required. In order to develop the method a different case scenario for the calculation of the maximum storage volume was assumed. This assumption was that the receiving water only provided a dilution capacity of 1:1. Consequently, a storage volume of 120l/head required to be implemented at the

overflows. As discussed in section 18.3 an assessment requires to be made of the likely storage volume which would be utilised to protect against flooding. This requires to be carried out prior to analysing overflow performance to ensure accurate spill volumes are obtained. The flood storage analysis is discussed in detailed in sections 18.5 and 18.6.

18.5 Flood Analysis

In terms of flood protection, a general rule that the city centre areas of a drainage catchment should be protected from flooding against rainfall events of return periods up to 5 years, and that out lying areas should be protected against events with return periods of up to 2 years was used. The Perth sewerage hydraulic evaluation (**Jack, 1995**) showed that the thirty minute duration design event was critical for the system. Figure 18.1 shows the locations of flooding which occurred under the 5 year 30 minute event.

18.6 Locations of Storage to Prevent Flooding

Unlike CSO storage, it is apparent that a significantly greater number of storage volumes are required to prevent flooding incidences.

Fig. 18.1 Flood Locations – Perth (5yr 30min event)



From the analysis it was deduced that a total storage volume of **2130m³** would be required to protect the catchment entirely from flooding. However, after considerable analysis and discussions with NoSWA, it was concluded that the majority of flooding incidents which occurred in the system were a consequence of various hydraulic throttles resulting from the system geometry. Consequently, storage was not deemed to be the most appropriate upgrading measure. The Bridgend sub-catchment, to the east of the River Tay, was an exception and would have benefited from storage, however, no appropriate location existed. This was due to the steep gradients in the catchment. NoSWA's intended solution to the flooding problem was therefore to improve screening methods to remove ragging on the overflow screens at the main overflows within the system and provide pumps to ensure free discharge of the overflowing sewage. This would alleviate the backing up problem which exists in the current system and thus reduce the flooding incidences. The adverse affect of this proposal was that a significantly greater spill volume would be discharged to the River Tay. However, the abundant assimilative capacity of the River Tay made this a feasible option. The possibility of NoSWA utilising storage in the future cannot be ruled out entirely, although it can confidently be stated that if storage is utilised the volume would not be significant, due simply to the practical constraints discussed and the de-ragging/pumped overflow solution which is to be adopted. Consequently, it was decided to evaluate overflow performance for the purpose of total emission analysis using the unmodified system. The reason being that if a small flood storage volume was implemented it would not significantly affect the results as the consequential increased spills from the overflows and/or the increased loading to the WTP would also be small.

18.7 Overflow Performance Evaluation

The overflows in the Perth system which discharge to the River Tay are located at the Willowgate, South Inch and Friarton pumping stations (fig. 6.1). The evaluation of their performance was carried out using Method One Analysis. The Method One process (**Henderson and Dempsey, 1990**) involves regressing spill volume against rainfall characteristics. Twenty TSR events were utilised to

provide the necessary characteristics and the corresponding spill volumes. The regression analysis process is demonstrated in table 18.2 using the South Inch Pumping Station as the example:-

Table 18.2 South Inch Pumping Station Overflow - Spill Volumes

TSR	Rainfall Depth	Duration	Max. Hourly Intensity	Mean Intensity	Predicted Spill Volume from Hydroworks - PM
Event No.	R_d (mm)	D_u (hrs)	(mm/hr)	I (mm/hr)	(m ³)
1	8.880	1.5	11.98	8.745	1923
2	7.986	1.33	9.83	7.176	1480
3	14.628	4.67	11.35	8.286	3468
4	11.307	7.5	8.59	6.271	1564
5	4.642	1.17	6.35	4.636	361
6	4.781	1.33	6.52	4.760	445
7	4.715	3.17	5.57	4.066	426
8	3.584	1.5	4.79	3.497	198
9	10.752	6.33	4.89	3.570	914
10	4.876	5.17	3.73	2.723	56
11	11.599	7	6.19	4.519	912.2
14	22.885	7.67	8.2	5.986	6932
15	4.802	6.33	3.34	2.438	0
16	4.847	4.17	4.59	3.351	258
23	8.497	5.67	5.46	3.986	749
25	3.752	6.67	2.95	2.154	0
28	6.854	6.83	2.66	1.942	0
30	5.132	4.67	2.87	2.095	0
32	5.380	7.17	2.62	1.913	0
45	4.066	4.5	3.25	2.373	0

Regressing the spill volume against rainfall characteristics; duration of rainfall event (D_u), depth (R_d) and mean intensity (I) the following constants were obtained:-

Intercept = - 639.533, $a = 388.617$, $b = -216.2$, $c = -82.363$

Thus, based upon the twenty observations shown in the table, the following regression equation was produced:-

$$\text{Predicted Spill Volume} = (388.6174 \cdot R_d) + (216.2 \cdot D_u) - (82.363 \cdot I) - 639.533$$

An R^2 value of **0.919855** was obtained, thus demonstrating a high correlation between spill volume and the regressed variables. This process was repeated for the Willowgate and Friarton Pumping Stations. The developed regression equations for these respective overflows are shown overleaf:-

Willowgate Pumping Station

$$\text{Predicted Spill Volume} = 82.82371 R_d - 55.5756 D_u - 44.94I + 74.37581$$

$$R^2 = 0.939$$

Friarton Pumping Station

$$\text{Predicted Spill Volume} = 282.422 R_d - 95.077 D_u + -1.9454 I - 635.079$$

$$R^2 = 0.962$$

After these equations were generated, ten years of statistical rainfall data were used to ascertain the long term spill frequency of the respective overflows. The spill frequencies are shown in table 18.3.

Table 18.3 Willowgate Pumping Station Overflow Analysis

Spill Range (m ³)	YEAR										Total Spills	No. of events Spilling (> x m ³)	% of events in range (> x m ³)
	'83	'84	'85	'86	'87	'88	'89	'90	'91	'92			
1 to 500	115	96	121	113	113	112	95	100	118	102	1086	>0:- 1164	100
501 to 1000	5	7	6	8	6	8	4	3	6	6	59	>500:- 79	6.7
1001 to 1500	1	2	2	1	1	0	0	0	1	1	9	>1000:- 20	1.7
1501 to 2000	1	0	0	0	0	0	0	1	0	0	2	>1500:- 11	0.9
2001 to 2500	1	0	0	0	1	1	1	1	0	0	5	>2000:- 9	0.7
2501 to 3000	0	0	0	0	0	0	0	1	0	0	1	>2500:- 4	0.34
3001 to 3500	0	0	0	0	0	1	0	0	0	1	2	>3000:- 3	0.25
3501 to 4000	0	0	1	0	0	0	0	0	0	0	1	>3500:- 1	0.08
4001 to 4500	0	0	0	0	0	0	0	0	0	0	0	>4000:- 0	0

Table 18.3 shows that the majority of the spills fall in the range of 0-500m³. Only 6.7% of the events spill greater than 500m³.

Table 18.4 South Inch Pumping Station Overflow Analysis

SPILL RANGE (m ³)	YEAR										Total Spills	No. of events Spilling (> x m ³):-	% of events in range (> x m ³)
	'83	'84	'85	'86	'87	'88	'89	'90	'91	'92			
1 to 500	30	17	17	16	9	9	15	11	13	12	149	>0:- 1170	100
501 to 1000	24	15	18	12	16	12	12	21	16	17	163	>500:- 1021	87.339
1001 to 1500	13	11	20	12	14	11	13	11	23	17	145	>1000:- 859	73.481
1501 to 2000	4	2	3	5	1	8	4	4	1	4	36	>1500:- 715	61.163
2001 to 2500	17	5	11	11	10	15	13	15	16	9	122	>2000:- 679	58.083
2501 to 3000	6	6	8	6	11	18	16	12	12	8	103	>2500:- 557	47.647
3001 to 3500	4	9	8	11	12	13	5	10	10	5	87	>3000:- 454	38.836
3501 to 4000	3	7	6	5	10	7	7	8	7	10	70	>3500:- 367	31.394
4001 to 4500	5	6	6	3	7	4	4	5	7	5	52	>4000:- 297	25.406
4501 to 5000	3	4	3	3	5	2	0	4	2	1	27	>4500:- 245	20.958

Table 18.4 (Cont'd)

SPILL RANGE (m ³)					YEAR							Total Spills	No. of events Spilling (>x m ³):-	% of events in range (> x m ³)
5001 to 5500	4	3	3	10	5	5	2	3	5	5		45	>5000:- 218	18.648
5501 to 6000	0	3	3	0	4	4	4	2	1	1		22	>5500:- 173	14.798
6001 to 6500	0	2	5	4	3	1	2	1	2	1		21	>6000:- 151	12.917
6501 to 7000	2	3	2	2	0	3	1	2	3	0		18	>6500:- 130	11.121
7001 to 7500	0	2	5	0	1	1	2	1	0	3		15	>7000:- 112	9.581
7501 to 8000	1	2	2	1	2	0	0	2	2	2		14	>7500:- 97	8.297
8001 to 8500	0	1	0	1	0	1	0	1	1	0		5	>8000:- 83	7.100
8501 to 9000	1	0	0	0	3	1	0	1	1	2		9	>8500:- 78	6.67
9001 to 9500	0	1	1	0	0	2	0	1	2	0		7	>9000:- 69	5.902
9501 to 10000	0	0	1	1	1	1	0	1	0	1		6	>9500:- 62	5.303
10001 to 11000	1	2	0	2	1	1	1	0	0	2		10	>10000:- 56	4.790
11001 to 12000	0	0	2	2	1	0	0	0	1	1		7	>11000:- 46	3.935
12001 to 14000	3	2	1	0	3	0	0	1	1	2		13	>12000:- 39	3.33
14001 to 16000	0	0	2	0	0	0	0	0	0	1		3	>14000:- 26	2.22
16001 to 20000	0	0	2	3	0	2	0	1	0	0		8	>15000:- 23	1.967
20001 to 40000	2	3	1	0	1	3	1	2	0	2		15	>20000:- 15	1.283

The analysis of the South Inch pumping station overflow (table 18.4) showed that significantly greater spills are discharged from this overflow than at the Willowgate pumping station. It was shown that only 6.7% of the events in a ten year period spill more than 500m³ at Willowgate, whereas at the South Inch 87 % of the events spill greater than 500m³.

Table 18.5 Friarton Pumping Station Overflow Analysis

SPILL RANGE					YEAR							TOTAL SPILLS	No. of events Spilling (> x m ³)	% of events in Range (> x m ³)
(m ³)	'83	'84	'85	'86	'87	'88	'89	'90	'91	'92				
1 to 500	57	32	40	29	30	26	31	36	34	29		344	>0:- 1157	100
501 to 1000	27	16	25	22	20	25	25	23	29	28		240	>500:- 813	70.26793
1001 to 1500	12	11	12	13	24	24	20	17	22	7		162	>1000:- 573	49.52463
1501 to 2000	0	2	3	2	1	7	0	3	1	4		23	>1500:- 411	35.5229
2001 to 2500	6	13	10	9	13	12	11	11	12	16		113	>2000:- 388	33.535
2501 to 3000	7	9	9	3	7	6	0	9	6	3		59	>2500:- 275	23.76837
3001 to 3500	2	3	3	7	6	7	5	4	5	3		45	>3000:- 216	18.66897
3501 to 4000	3	4	5	7	7	3	2	1	3	4		39	>3500:- 171	14.7796
4001 to 4500	1	4	4	1	1	1	2	2	4	2		22	>4000:- 132	11.40882
4501 to 5000	0	2	6	0	4	3	2	4	1	1		23	>4500:- 110	9.507347
5001 to 5500	1	2	1	1	1	2	0	1	3	2		14	>5000:- 87	7.519447
5501 to 6000	1	1	0	1	1	1	0	2	1	2		10	>5500:- 73	6.309421
6001 to 6500	1	0	2	1	0	1	0	0	1	2		8	>6000:- 63	5.445117
6501 to 7000	1	2	2	1	1	1	0	0	1	2		11	>6500:- 55	4.753673

Table 18.5 (Cont'd)

SPILL RANGE	YEAR										TOTAL SPILLS	No. of events Spilling (> x m ³)	% of events in Range (> x m ³)
(m ³)	'83	'84	'85	'86	'87	'88	'89	'90	'91	'92			
7001 to 7500	1	0	0	2	3	0	1	0	0	2	9	>7000:- 44	3.802939
7501 to 8000	0	2	1	0	0	0	0	0	1	0	4	>7500:- 35	3.025065
8001 to 8500	1	0	1	0	1	0	0	1	0	0	4	>8000:- 31	2.679343
8501 to 9000	0	0	1	1	0	1	0	0	0	1	4	>8500:- 27	2.333621
9001 to 9500	0	0	0	0	1	1	0	0	0	0	2	>9000:- 23	1.9879
9501 to 10000	0	0	2	1	0	0	0	0	0	0	3	>9500:- 21	1.815039
10001 to 11000	0	0	1	1	1	0	0	2	0	0	5	>10000:- 18	1.555748
11001 to 12000	0	0	0	0	0	0	0	0	0	0	0	>11000:- 13	1.123596
12001 to 14000	2	1	0	0	0	2	1	1	0	1	8	>12000:- 13	1.123596
14001 to 16000	0	0	0	0	0	0	0	0	0	0	0	>14000:- 5	0.432152
16001 to 20000	0	1	0	0	0	0	0	0	0	0	1	>15000:- 5	0.432152
20001 to 40000	0	1	1	0	0	1	0	0	0	1	4	>20000:- 4	0.345722

Using the 75th percentile spill range as a reference for comparing the spill volumes at South Inch and at Friarton it can be seen that at the South Inch pumping station, 75% of the events spill less than 4,000m³ whereas at Friarton, 75% of the events spill less than 2,436m³ (interpolated). Consequently, it is observed that the South Inch overflow discharges a significantly larger volume of sewage to the receiving watercourse. The analysis therefore showed that the utilisation of storage at the Bridgend subcatchment would make no fundamental difference to the system performance, as the volumes spilled to the receiving watercourse from this overflow are insignificant in comparison to the volume spilled from South Inch and Friarton i.e. a storage volume of only 500m³ would prevent 87% of all the events in a ten year period from spilling. Consequently, the useful locations for storage in the system would be the South Inch pumping station and the Friarton Pumping Station. The required maximum storage volumes for the respective overflows were therefore calculated utilising the SDD method with the +100% safety factor. Following SDD guidelines a storage requirement of 120l/head is necessary for a receiving water course which provides a dilution ratio of 1:1 (hypothetical case). As the population of Perth upstream of the South Inch Pumping station is approximately 41,000 a storage volume of 4920m³ is calculated. Applying the correction factor of +100% a maximum storage volume for the South Inch pumping station would be approximately 10,000m³. From analysis of the Method One table it can be seen

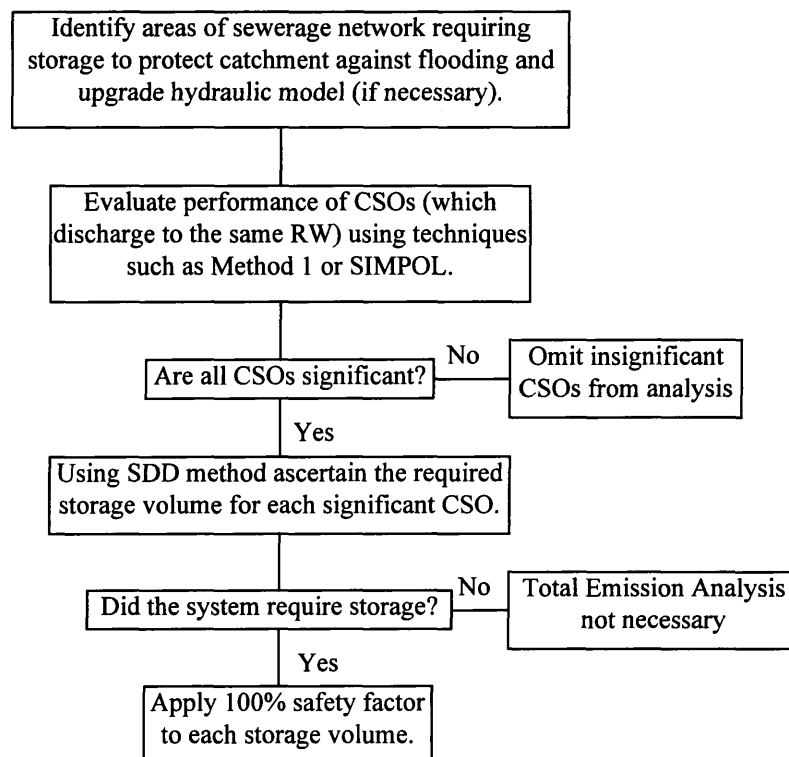
that this is a suitably conservative volume as only a very small proportion of the events (4.7%) in ten years will be expected to spill more than this. As the Friarton Pumping Station is two hundred metres downstream of the South Inch Pumping Station on the same interceptor sewer, the storage volume provided at the South Inch would ordinarily mean that storage would not be required at the Friarton Pumping Station. However, as an additional subcatchment connects to the system between these two overflows an additional storage volume was calculated to account for the extra flows which this subcatchment would produce. The connecting subcatchment held an estimated population of approximately 1200, and thus following the SDD criteria a storage volume of 144m³ would be required. Applying the +100% correction factor, the maximum storage volume required for the Friarton Pumping Station was calculated as approximately **300m³**. The maximum storage volumes for the two main overflows are shown below:-

South Inch:- 10,000m³

Friarton:- 300m³

As the storage volume required at Friarton overflow is so small in comparison to the storage volume required at the South Inch it was decided not to utilise this storage volume as the analysis would not be affected. This is emphasised by the factor of safety applied at the South Inch pumping station which provides an additional 5000m³ of storage. The maximum storage requirements for the Perth system was therefore defined as **10,000m³**, located at the South Inch overflow. Figure 18.2 summarises the general procedure for obtaining the largest possible storage volume.

Figure 18.2 Largest Possible Storage Volume (Flow Chart)



18.8 Summary

The objective of this chapter was to determine the maximum possible storage volume which should be used for the total emission analysis. It was discussed that this storage volume should not be chosen arbitrarily but be based on site specific characteristics such as CSO spill performance and the receiving waters assimilative capacity. A Method One or SIMPOL type analysis was suggested to ascertain the overflows within the system which would benefit from storage. A method to size the storage volume was also proposed, based upon the SDD method, albeit with a + 100% correction factor to ensure the reference TEAP was obtained. Utilising the Method One analysis and the modified SDD approach it was deduced that only one location in the Perth system would benefit from storage (the South Inch Pumping Station Overflow) and that the “largest possible” storage volume which should be used for the total emission analysis would be 10,000m³. The determination of this value allowed the reference TEAP to be calculated (chapter 19).

Chapter Nineteen

Total Emission Analysis Period

19.1 Introduction

The next stage in the method was the calculation of the Total Emission Analysis Period. The principal objective of this chapter was to exemplify the method of calculating the WTP disruptive period, to obtain the TEAP, for both BOD and Ammonia. This chapter therefore discusses the principles and procedures which were adopted to calculate the TEAP for the Perth system.

19.2 Worst Case Rainfall Profiles

Utilising the STORMPAC historical rainfall profiles, a table was produced showing the predicted volumes of rainfall for each month in a ten year period. The profiles were screened to remove events with characteristics of less than 1mm volume, 0.2mm/hr peak intensity and 0.2mm/hr average intensity. Such events would not give rise to large volumes of runoff and were therefore not of critical importance with respect to CSO spill or WTP disruption. The monthly totals are shown in table 19.1. Bold denotes the month(s) of the year in which the greatest amount of rainfall fell.

Table 19.1 Monthly Rainfall Totals (1970 - 1979)

Year/ Month	<i>Jan</i>	<i>Feb</i>	<i>Mar</i>	<i>Apr</i>	<i>May</i>	<i>Ju n</i>	<i>Jul</i>	<i>Aug</i>	<i>Sep</i>	<i>Oct</i>	<i>Nov</i>	<i>Dec</i>
1970	105	13	53	9	65	25	53	46	37	81	55	103
1971	93	29	24	104	66	35	37	8	144	73	42	42
1972	56	24	47	26	20	91	34	53	67	36	51	91
1973	128	24	21	44	29	80	11	101	103	62	50	64
1974	41	44	73	82	33	128	101	29	52	36	60	73
1975	120	39	20	26	30	27	28	48	28	74	52	62
1976	59	27	47	37	42	14	32	37	51	92	46	80
1977	110	24	35	63	61	20	64	35	27	46	27	3
1978	55	45	71	54	24	50	85	73	34	34	70	109
1979	47	32	53	25	58	100	53	92	52	103	60	97

Utilising 1970 as an example it can be seen that the wettest months over the summer and winter periods are May, July, January and December, with the worst case summer month being May, and the worst case winter month being January. With respect to WTP performance, the winter months are the most critical and consequently the example analysis has been carried out using the January rainfall profile. This is the wettest month in the entire year and consequently would allow the total emission problem to be thoroughly evaluated. Table 19.2 shows the days within this month where rainfall occurred:-

Table 19.2 Days of Rainfall in Wettest Month

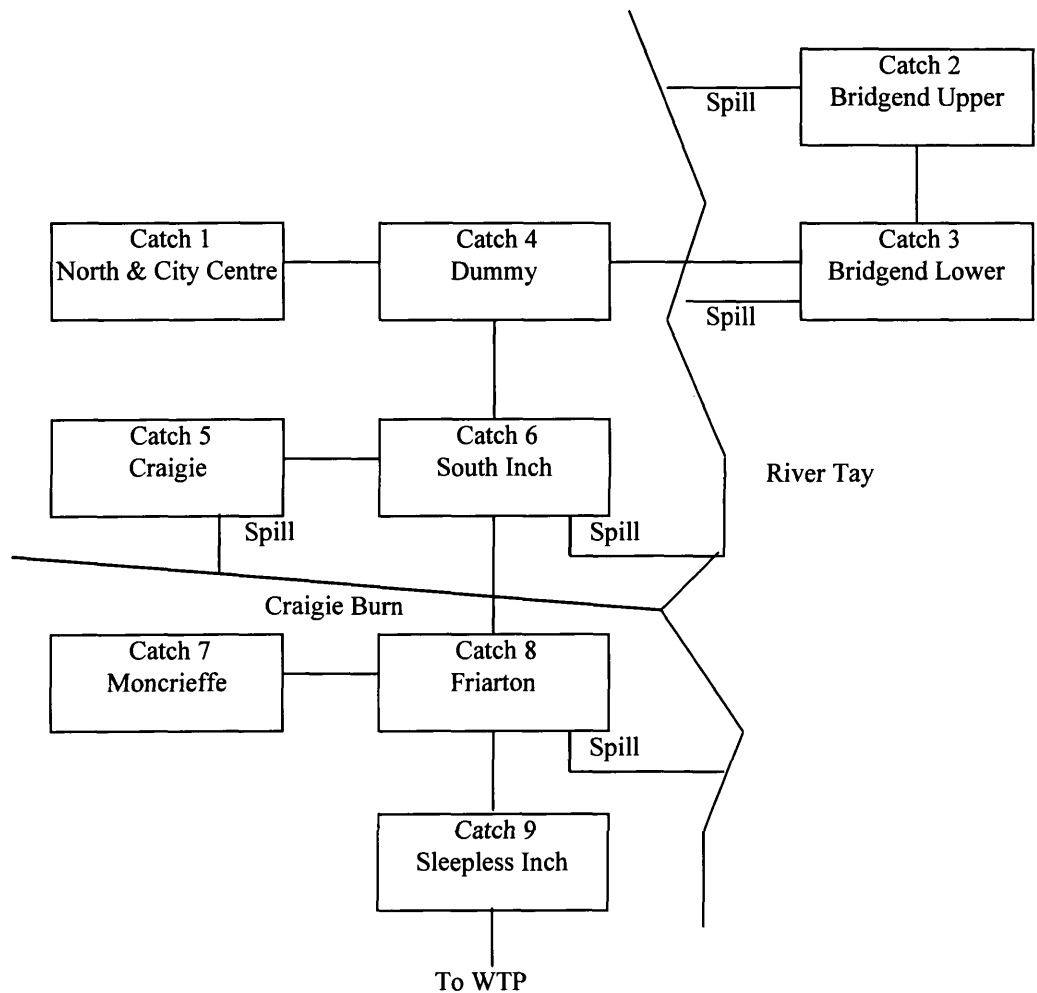
Jan '70 Date	Duration (hrs)	Depth (mm)	Mean Int (mm/hr)	Max. Int. (mm/hr)	Start Time (hr)
1	13	3	0.2	0.4	04:00
2					
3					
4					
5					
6	20	32.4	1.6	5.3	15:00
7	14	14.8	1.1	1.8	14:00
8	5	11.5	2.3	2.4	12:00
8	4	2.3	0.6	0.9	19:00
9	10	9.8	1	2.9	4:00
10					
11					
12	4	1.5	0.4	0.6	18:00
13	3	1.2	0.4	0.6	19:00
14					
15	8	7.2	0.9	2.6	18:00
16	3	1.3	0.4	0.7	07:00
17					
18	1	2.3	2.3	2.3	4:00
19					
20	4	3.1	0.8	2	8:00
20	9	5.4	0.6	1.8	14:00
20	3	1.3	0.4	0.8	24:00
21					
22					
23	4	1	0.3	0.4	22:00
24	5	2.5	0.5	1.2	3:00
24	8	5.3	0.7	2	14:00
24	3	1.2	0.4	0.5	24:00
25					
26	2	1	0.5	0.5	04:00
27					
28					
29					
30	4	5.2	1.3	2.1	6:00
30	4	3.7	0.9	1.6	18:00

As discussed in chapter sixteen, it is only necessary to carry out the total emission analysis over the wettest period within this month as this period would provide the worst case problems at the WTP and thus the greatest (potential) total emission problem. It can be seen from table 19.2 that the worst period begins on the 6th of January 1970. In order to calculate the TEAP, the WTP influent characteristics resulting from these events required to be defined to ascertain firstly, the duration of combined loading to the plant, and secondly, the duration required before the plant could restore itself to normal functioning.

19.3 SIMPOL

Due to the very large simulation times associated with complex modelling the WTP input data was provided using the UPM simplified hydraulic model, SIMPOL. The principle adopted within SIMPOL is to simplify the modelling requirements via conceptual representations (referred to as 'tanks') of the salient components in the system (**FWR, 1994**). Calibration is carried out by comparing the spill volume and continuation flow from these tanks with those predicted by the full deterministic model. The process therefore requires the utilisation of various rainfall events of different characteristics to allow the overall response of the drainage catchment to be represented. SIMPOL was only used to ascertain the hydraulic response of the Perth drainage system. No attempt was made at constructing a simplified quality model. A diagrammatic representation of the constructed SIMPOL hydraulic model for the Perth drainage system is shown in figure 19.1:-

Figure 19.1 SIMPOL Representation of the Perth Sewerage System



19.31 Calibration

Calibration of SIMPOL was achieved by running various rainfall events through the full hydraulic model and inputting the spill volume and continuation flows for each of the catchments/'tanks' into SIMPOL. The package then calculates the maximum continuation flows for each of the catchments/tanks in the system. The calibrated values are shown in table 19.3.

Table 19.3 SIMPOL Calibrations

SUB-CATCHMENT/ 'TANK'	CONTINUATION FLOW (l/s)
1 - North & City Centre	2700
2 - Bridgend Upper	98
3 - Bridgend Lower	78
4 - Dummy	4000
5 - Craigie	530

Table 19.3 (Cont'd)

SUB-CATCHMENT/ 'TANK'	CONTINUATION FLOW (l/s)
6 - South Inch	505
7 - Moncrieffe	1800
8 - Friarton	560
9 - Sleepless Inch	1200

It was noted that the calibrated continuation flows as defined by SIMPOL for the South Inch and Friarton pumping stations tanks were low (505l/s and 560l/s, respectively) relative to the capacity of the pumping stations (1000l/s) and the downstream sewers. This anomaly was investigated and was found to be a consequence of the on/off operation of the pumps. Substantial volumes of sewage are spilled from the South Inch Pumping Station and consequently, the pumps do not constantly pump to their full capacity for long durations. As SIMPOL averages the pass-forward flows over an hourly interval the maximum continuation flows, as calculated by SIMPOL are therefore less than the instantaneous peak flows which actually result. As pump operation can significantly affect the pass-forward flow rate a test was carried out to ascertain whether the draining of the storage tank would increase the frequency of pump operation and thus the calculated hourly average continuation flow. The results from the analysis showed this would be the case and that the maximum average hourly pass forward flow would be closer to 950l/s. It was also observed however, that as the pass-forward flows were increased, greater volumes of sewage were discharged from the downstream overflow at Friarton. This is demonstrated by table 19.4.

Table 19.4 Pumping Station Spills

	South Inch Pumping Station (Spill Vol. m ³)	Friarton Pumping Station (Spill Vol. m ³)
Storage:- 0m ³	14,571	6621
Storage:- 10,000m ³	4,571	9165

It can be seen that if the full capacity of the downstream pipe is utilised to drain the storage tank, the spill volume from the Friarton Pumping station will be increased by 2,545m³. Consequently, the full benefit of the storage would not be obtained. As it is an objective of rehabilitation to ensure that the implemented

solution should not cause additional problems elsewhere in the catchment it was necessary to limit the pumping pass-forward rate to a level which would not increase the spills from Friarton pumping station overflow. Consequently, the maximum pass forward rate was kept at the calibrated value of 505l/s. As the pass-forward rates affect spill volumes from the CSOs and the loading to the WTP it can be deduced that total emissions can be further optimised by real time control of the existing storage within the system. This is discussed in more detail by **Guderain et al, (1997)**, who states that a maximum continuation flow will exist which minimises the total emissions. This however is of relevance to real time control projects which are concerned with optimising emissions from a system already containing storage. The objectives of this research project were fundamentally different, as the aim was to ascertain the effect *varying* storage volumes have with respect to total emissions. The additional optimisation via real time control is out-with the scope of this project but is being considered via another study currently being carried out at Imperial College, (**Schutz, 1998**).

19.4 SIMPOL Analysis (Jan'70)

The 10,000m³ of storage was input into the unmodified SIMPOL model and the January '70 rainfall profiles were run through the model. As the version of SIMPOL which was available for use within the project was not the continuous simulation model, adjustments required to be made to the output results to account for this limitation. This was necessary because SIMPOL assumed the storage to be empty at the beginning of each event whereas it was apparent that for certain events this assumption was not true. The spill volumes and continuation flows were recalculated by taking into account the volume of stored wastewater at the onset of subsequent events and by considering the draining/filling process of the tank. With reference to the pass-forward flows, volume was drained from the tank when spare capacity was available in the downstream pipe i.e. the maximum carry on flow at the South Inch Pumping station was calibrated as being 1818m³/hr; for each hour where the continuation flow dropped below this value a volume of stored wastewater would be drained to ensure the full continuation flow of 1818m³/hr was utilised. This approach

therefore provided more realistic spill volumes and more realistic WTP continuation flows for the respective events. Table 19.5 highlights the under prediction of CSO spills which would have resulted if these adjustments were not made. The table presents the CSO discharge loads over the worst period in the monthly rainfall profile (6/1 to 9/1).

Table 19.5 CSO Discharge Loads

Scenario:- Om ³ Storage	CSO Spill Volume from 6/1/70-9/1/70 (m ³)
SIMPOL Discrete Simulation	66,315
Adjusted SIMPOL Results	81,057
Difference	14,742 (+22%)

It can be seen that CSO spills would have been under predicted by 14,742m³ if these adjustments had not been made.

19.5 TEAP Assessment

With respect to the scenario utilising 10,000m³ of storage the rainfall events occurring between the 6/1/70 to the 9/1/70 placed the WTP under full hydraulic loading for a period of 91 hours (approximately, 4 days). In order to define the TEAP, WTP analysis was carried out to ascertain whether the plant would restore itself to normal functioning by the onset of the subsequent events (which began on the 12/1/70). As it was not the objective of the work at this stage to carry out the actual total emission analysis, the exact input hydrographs as dictated by SIMPOL were not utilised in the WTP analysis tests. The tests were carried out using only the average flow characteristics which the combined flows caused. Utilising variations of these characteristics an assessment was also made with regard to the TEAPs which would be required if the average influent characteristics varied from those defined above. This was important as the various periods throughout the year, which required to be analysed to ensure statistical compliance with the discharge consents, could not be expected to produce the same WTP influent profiles. By carrying out comprehensive sensitivity testing at this stage, a table could be produced which would show the disruptive periods which result from a variety of different ‘average’ influent characteristics. This would therefore prevent WTP performance tests having to be

carried out each time a different rainfall period is being analysed. The rainfall events occurring between the 6/1 and the 9/1/70 were observed to produce combined flows in the range of 555l/s to 660l/s. This resulted in an average WTP inflow of 600l/s being calculated. This flow was therefore fed to the WTP model over the 4 days period. It was shown in chapter fifteen that the associated wastewater quality characteristics corresponding to a flow rate of 600l/s were 150:100:10mg/l, TSS:BOD:Amm. The results for Flow, TSS and BOD are shown in figures E.29 and E.30. For clarity, ammonia is considered later. Figures E.29 and E.30 show one day of dry weather influent prior to the four days of combined loading (hrs 0-24), and three days of dry weather influent subsequent to these flows (hrs 120 - 196). This was included to allow the plant performance, before and after the events, to be compared. It can be seen clearly from figures E.29 and E.30 that the results that the four day combined loading period does not cause a period of continued disruption at the plant. This is concluded as the plant discharges effluent of the same standard as noted between hours 0-24 (DWF). In order to ascertain the disruptive periods corresponding to different influent characteristics the following additional tests outlined in table 19.6 were carried out:-

Table 19.6 WTP Test Data

Test No.	Flow Rate (l/s)	Duration (dys)	Associated Quality Characteristics (TSS:BOD)
1b	600	6	150:100
1c	600	8	150:100
2a	800	4	140:50
2b	800	6	140:50
2c	800	8	140:50
3a	1000	4	125:20
3b	1000	6	125:20
3c	1000	8	125:20

19.51 Results

Table 19.7 presents the results obtained from the above analysis.

Table 19.7 WTP Disruptive Periods

	WTP Loading Duration:- 4 days	WTP Loading Duration:- 6 days	WTP Loading Duration:- 8 days
Flow Rate:- 600l/s	No additional disruption	No additional disruption	No additional disruption
Flow Rate:- 800l/s	No additional disruption	No additional disruption	No additional disruption
Flow Rate:- 1000l/s	Disruption - BOD	Disruption - BOD	Disruption - BOD

Table 19.7 shows that additional disruption occurred only with an influent flow rate of 1000l/s. A graphical representation of the WTP performance with respect to test 1c and 2c are provided in figures E.31 to E.36, demonstrating the plants rapid return to normal functioning.

It can be seen from figures E.31 to E.36 that the WTP returns to normal functioning as soon as the dry weather flows re-establish. Consequently, the required TEAP for events which produce similar influent characteristics would be the period which the plant is subject to combined loading i.e. no extended analysis period would be required.

19.6 Periods of Continued Disruption

The reason a plant can experience periods of continued disruption after the flows have returned to dry weather levels is because of insufficient bacteria being present within the reactor to optimally treat the influent wastewater. This can result from two main problems; the first being a loss in biomass due to sludge loss from the final settlement tank, or secondly, bacteria reduction as a consequence of limited substrate within the influent.

Figures E.31 to E.36 show that neither of these problems occurred during tests 1c and 2c. This explains why the plant did not experience continued disruption. Nevertheless, the analysis has provided significant information with respect to the plant operation under combined loading; firstly, with reference to effluent TSS concentrations it can be seen from figs E.32 and E.35 that the effluent quality deteriorates with the duration of combined loading. This reflects the

comments made by **Harremoes et al, 1993** who stated that ‘maximum flow conditions cannot be dealt with by clarifiers for sustained periods. Eventually a failure will occur’. Although reference was being made to shallow settler tanks, the principle is shown clearly by the aforementioned graphs. An additional test were carried out to ascertain if and when failure would occur.

19.7 Sludge Blanket Breakthrough Analysis

In order to carry out this test the plant was placed under a combined load of 600l/s (150mg/l TSS) for a period of twenty five days. These influent characteristics were chosen as they provide the worst case loading with respect to the final clarifier i.e. an influent of 1000l/s would place the clarifier under the same hydraulic conditions (due to the three dry weather flow split) however, the associated influent would be more dilute (120mg/l). The results from this analysis can be seen in figures E.37 and E.38. Figure E.38 shows that TSS quality does not continue to decrease with time as an equilibrium concentration is eventually reached. Consequently, even under these extreme loading conditions no ‘breakthrough’ (or ‘sludge loss’) was observed. This correlates to the statement that this problem is related to shallow clarifiers which do not have sufficient depth to accommodate the increases in sludge blanket levels. As the final clarifiers at Sleepless Inch are deep (3.36m) the ‘breakthrough’ phenomenon was not observed. The second point of interest was with respect to the biomass concentration within the reactor during and after the combined loading events. From figs. E.33 and E.36 it can be seen that a significant proportion of biomass was removed from the tank at the onset of the combined flows. In chapter two it was explained that this was a consequence of displacement due to the high hydraulic load. In test 1c, which had influent characteristics of 600l/s for 8 days, it can be seen that the biomass concentrations dropped from approximately 1000mg/l to approximately 450mg/l and rose steadily to 600mg/l by the end of the combined loading. The concentration of biomass at the end of the combined flow was noted to be of a sufficient magnitude to properly treat the influent dry weather BOD (i.e. no continued disruption). This was also the case for test 2c, which had influent characteristics

of 800l/s - 8 days) however, the figures showed a different trend with respect to the biomass concentrations. With reference to figure E.36 it can be seen that there was a more pronounced drop in biomass concentration with a greater reluctance for the biomass to increase with time. Consequently, this emphasised that the reduction in biomass concentration could not be solely due to hydraulic displacement as the 3 DWF setting prior to the activated sludge setting ensured the same flow rate was being applied to the reactor under both tests. The only difference with respect to the influent characteristics with respect to activated sludge tank is therefore the concentrations within the influent. Referring to chapter fifteen, the associated BOD concentrations for flow rates of 600l/s was defined as 100mg/l, whereas the associated concentration for flow rates of 800l/s was defined as 50mg/l. It is therefore apparent that, under long term combined loading, the influent substrate had an effect on biomass concentration and this explains why continued disruption was experienced in test 3. The performance of the plant with respect to these tests is shown in figures E.39 to E.47.

It can be seen that tests 3a, 3b and 3c all show the same pattern with respect to the continued disruption. Although the duration of combined loading in tests 'b' and 'c' are longer, the disruption at the plant is no greater than that which was observed in test 'a' (disruptive period = approximately 24 hrs, peak effluent BOD = approximately 40mg/l). The reason the plant does not experience greater disruption under tests 'b' or 'c' can be explained via figs E.41, E.44 and E.47 which show the resulting concentration of biomass within the reactor. It can be seen from these figures that the biomass does not continue to decrease with time but levels out at approximately 145hrs. Consequently, for each scenario the biomass concentration at the end of the simulation is at a similar level and thus the degree of disruption experienced at the plant is the same. From further observations it can be seen that the re-established dry weather BOD concentrations are suitably treated when the active biomass concentration increases to approximately 375mg/l (fig. E.47 - 240hrs). Consequently, from these observations it was hypothesised that flow rates of up 800l/s would not cause continued disruption at the plant, regardless of duration of combined loads. This was because the biomass concentrations were observed to increase to a level

above 375mg/l at the onset of the dry weather flows (fig. E.45 and fig. E.46). Furthermore, the general trend of the biomass, where diluted BOD influent concentrations were 50mg/l, or greater, was one of marginal increase. Consequently, the longer the duration of the combined load the less likely would be the possibility of the plant experiencing continued disruption. This was investigated by prolonging the combined flow to the plant at 800l/s for 20 days (these conditions would not actually occur in reality, however the test was carried out simply to provide more information with respect to the above hypothesis). The results are shown in figures 19.2 to 19.4.

Fig. 19.2 Influent of 800l/s for 20 days – WTP Effluent – Flow

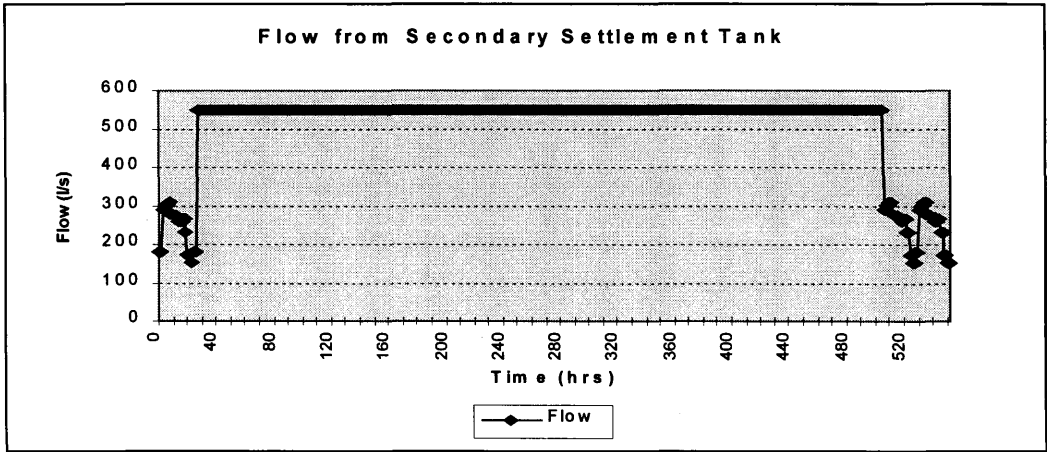


Fig. 19.3 Influent of 800l/s for 20 days – WTP Effluent - BOD

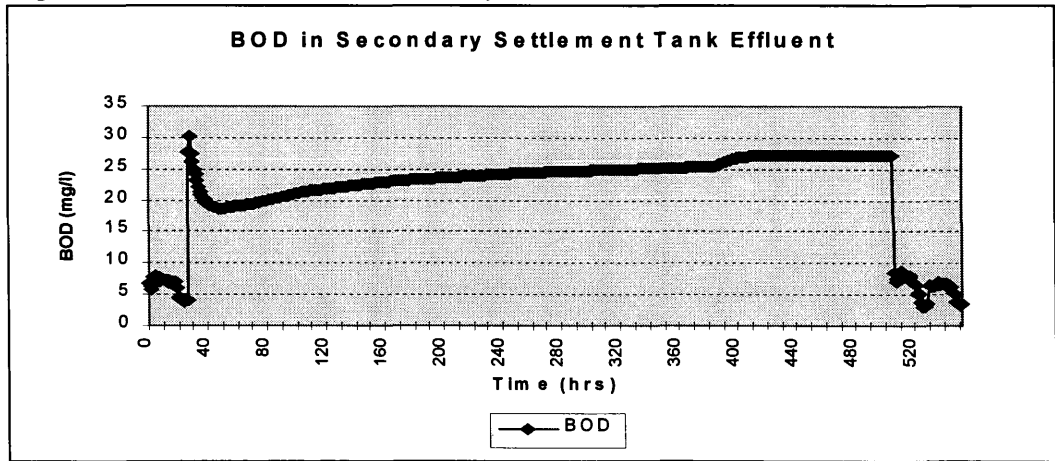
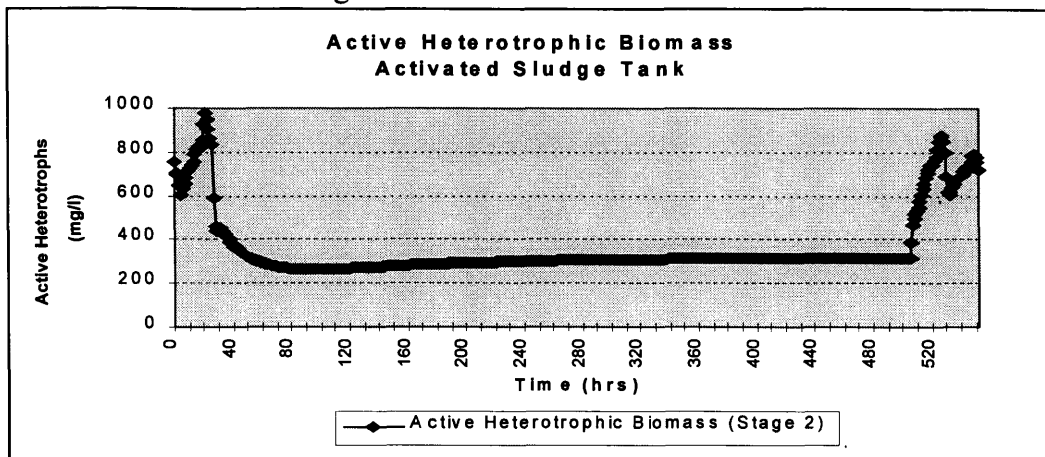


Fig. 19.4 Influent of 800l/s for 20 days - Heterotrophic Biomass in Activated Sludge Tank



It can be seen from figs. 19.2 and 19.3 that the plant, as expected, did not experience continued disruption after the dry weather flows re-established. This was because, as hypothesised above, the biomass concentration (fig.19.4) increased above the critical concentration at the onset of the dry weather flows. It was noted that the biomass increased towards this minimum concentration even during the period of combined loading and thus it can be confidently concluded that events generating flows of up to 800l/s, for any duration, would not cause continued disruption at the WTP.

Of additional interest was the performance of the plant with respect to 1000l/s. Although it had been shown that additional disruption would occur if the plant was loaded at this duration for four days, it was hypothesised from observation of the biomass concentrations that additional disruption could occur if the plant was subject to flows of 1000l/s for a period as little as one day. This can be observed from figures E.41, E.44 and E.47 which show that the biomass concentration very quickly fell below the critical threshold. Consequently, additional tests were carried out to ascertain the minimum duration which the plant would require to be subject to before disruption occurred. From figures E.48 to E.50 it can be seen that when the dry weather flows re-establish the concentration of active biomass increased to approximately the critical concentration of 375mg/l. However, it can also be noted that the biomass does not increase much beyond this level for approximately ten hours, thus resulting in a small period where the effluent quality is marginally poorer than it would have been under normal steady state

conditions. It was therefore concluded that the influent characteristics, as defined above, could give rise to a maximum disruptive period of approximately fifteen hours. This test was repeated, but with the combined loading duration extended to two days. With reference to the figures E.51 to E.53 it can be observed that the biomass concentrations increased only to approximately 205mg/l at the onset of the dry weather flows. As this is significantly lower than the threshold concentration of 375mg/l, poor effluent quality was produced (peak 25mg/l). A period of 24 hours was required before the plant could restore itself to normal functioning. Tests 3a-c showed that similar results would be obtained for combined loading durations of up to eight days as a consequence of the biomass concentrations reaching an equilibrium value at approximately 145hrs (fig. E.47). A further test was carried out to ascertain whether the biomass would remain at this equilibrium value (80mg/l) if the loading duration was increased beyond eight days. This was investigated as it was believed possible, from a visual interpretation of the graph, that the biomass may decrease further with longer combined loading periods. The plant was therefore loaded at 1000l/s for a period of twenty five days. The results are shown in figures E.54 to E.56.

It can be seen from figure E.56 that the biomass concentrations did not decrease with time, but actually increased. This was an unexpected trend and was attributable to the influent providing suitable quantities of substrate to allow cell maintenance and marginal growth, thus offsetting the biomass decay. A disruptive period of one day was however still observed to occur. It was therefore concluded that longer periods of disruption could only result if the plant was subject to influent with concentrations lower than 20mg/l. This is demonstrated in figures 19.5 to 19.7, which present the results obtained from a simulation where the plant was fed influent of 10mg/l for eight days.

Fig. 19.5 Influent of 1000l/s (@10mg/l BOD) for 8 days - WTP Effluent - Flow

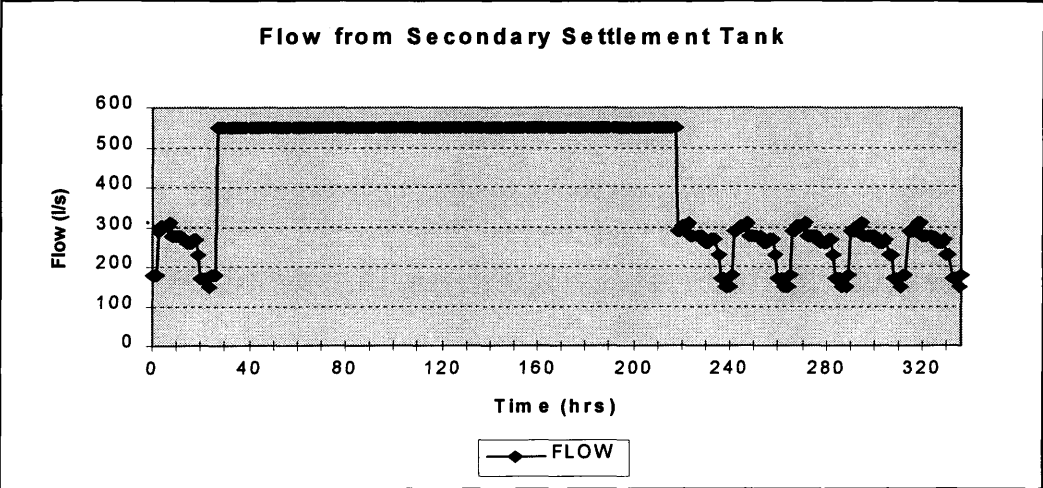


Fig. 19.6 Influent of 1000l/s (@10mg/l BOD) for 8 days - WTP Effluent - BOD

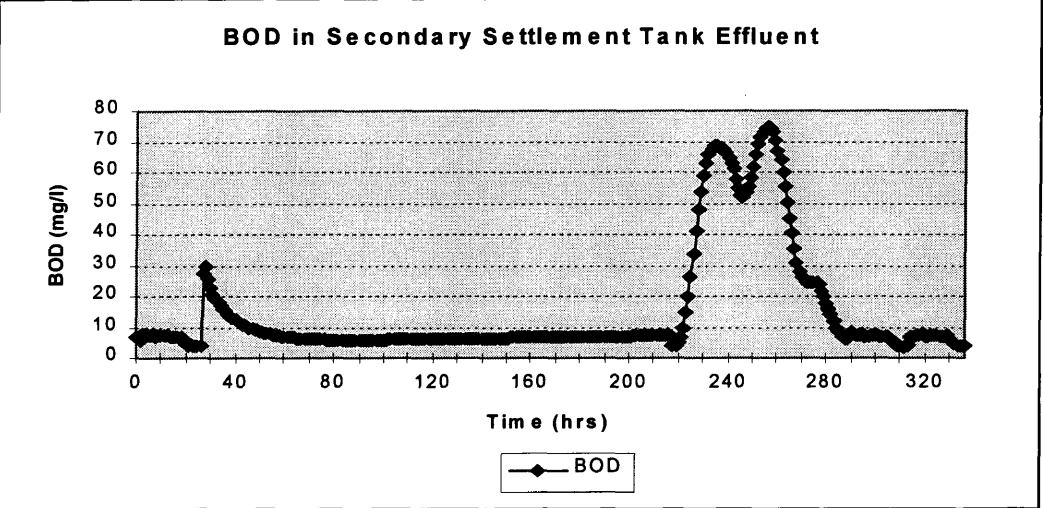
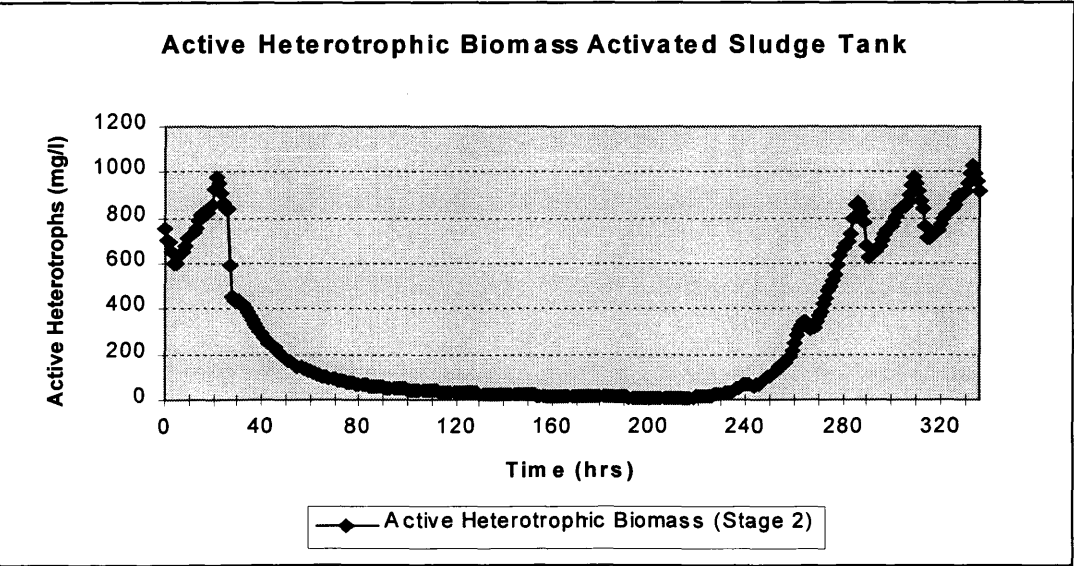


Fig. 19.7 Influent of 1000l/s (@10mg/l BOD) for 8 days - Heterotrophic Biomass in Activated Sludge Tank



Figures 19.5 to 19.7 show that with the very dilute influent BOD concentration (10mg/l), the bacteria exhibit a general trend of decreasing concentration with time. Consequently, only under these conditions would the disruptive period show a dependency on the duration of combined loading. However, it was apparent from the data collection exercise that such low concentrations would not be expected under normal combined loading conditions. Even from a mass balance perspective it was calculated that the maximum inflow of 1000l/s, could dilute the dry weather flows (which have average characteristics of 200l/s, 170mg/l) by a factor of 5. This would therefore provide a minimum influent BOD concentration of 34mg/l. Consequently, the utilisation of 20mg/l as the influent concentration was considered to be conservatively low. No justification could therefore be found for reducing the influent concentrations further and the maximum disruptive period which would exist at the plant was therefore taken as one day. Discussions with WTP experts at NoSWA (McQueen & James, 1997) confirmed the validity of the above results as it was the experience of the authority that the problem of bacteria 'die off' only occurs where very dilute influent is sustained for extensive periods. A typical example with respect to the Perth WTP was the January '93 floods when the sewer drained flood water from the river for a period of approximately two weeks. As these dilute flows were passed to the plant for extensive periods, the biomass were consequently killed. **Generally, however, the problem has never been observed to occur under normal operating conditions.** Other circumstances where biomass were observed to die off were at plants which treat sewage from small 'summer population' areas whose connecting sewerage drains a significant amount of infiltration. As the infiltration can dilute the small organic load being conveyed to the plant, the biomass concentrations can be adversely affected.

19.8 Summary

The analysis has shown that combined flows reduce the concentration of biomass within the reactor via two main processes. The first being the mechanical displacement of biomass as a result of the increased hydraulic load and the second being the further reductions in biomass due to limited substrate. The

analysis has shown however, that the reduction in biomass concentrations due to limited substrate would not cause significant problems at the plant unless very dilute influent concentrations occurred. The degree of dilution required to cause these problems cannot be expected to occur under normal loading conditions. Consequently, the maximum disruptive period at the plant was observed to be twenty four hours. Table 19.8 below summarises the disruptive periods (with respect to the BOD removal processes) which were ascertained for the varying influent characteristics:-

Table 19.8 WTP Disruptive Periods for BOD Analysis

Loading Duration	600l/s (150:100mg/l TSS:BOD) Disruptive Period	800l/s (140:50 mg/l TSS:BOD) Disruptive Period	1000l/s (125:20mg/l TSS:BOD) Disruptive Period
1hr	No Disruption	No Disruption	15hrs
2 hrs	No Disruption	No Disruption	24hrs
> 2 hrs	No Disruption	No Disruption	24hrs

19.9 WTP Disruptive Periods for Ammonia Analysis

The autotrophic bacteria, which are responsible for the nitrification of ammonia, have lower growth rates than the heterotrophic bacteria and would thus be expected to give rise to greater disruptive periods than those observed in table 19.8. Consequently, tests were carried out to ascertain the duration of continued disruption which would be expected for the nitrification process. As the WTP at Sleepless Inch was not designed to nitrify, synthetic modifications required to be made to the model to allow these tests to be carried out. In order to produce a model plant which would nitrify a wastewater, the 'Food to BioMass' (F_0/M) ratio required to be decreased. Lowering the F_0/M ratio increases the average time the activated sludge remains in the system (sludge age). This has the benefit of ensuring that the small quantities of autotrophic bacteria are not "washed out" of the WTP a consequence of the essential wasting process. An additional benefit of a lowered F_0/M ratio is an increase in reactor volume. This provides a greater hydraulic residence time and therefore promotes the possibility of the autotrophic bacteria successfully nitrifying the wastewater. The F_0/M ratio of the existing plant was calculated using the following data and equation 19.1:-

Peak Settled Sewage Flow (DWF)	=	24,192 m ³ /d
Peak Settled Sewage BOD (DWF)	=	0.160 kg/m ³
MLSS	=	1.65 kg/m ³
Vol. of Tank	=	3,600 m ³

$$\text{Tank Vol.} = [(\text{Flow}_{\text{settled sewage}} \times \text{BOD}_{\text{settled sewage}}) / (F_o/M)] / \text{MLSS.} \quad (\text{eqn. 19.1})$$

$$\begin{aligned} F_o/M &= (\text{Settled Sewage Flow} \times \text{BOD}) / (\text{MLSS} \times \text{Vol. of Tank}) \\ &= (24192 \times 0.16) / (1.650 \times 3600) \\ &= \underline{\underline{0.67}} \end{aligned}$$

The corresponding sludge age was calculated as 2.5 days. With reference to **Metcalf and Eddy, 1991** a F_o/M ratio of 0.15 would produce adequate nitrification of a typical wastewater. However, it was found that a suitable degree of nitrification would occur with a F_o/M ratio of 0.18. The corresponding sludge age was calculated as 4.5 days. Figure E.57 and E.58 demonstrate the plants ability to nitrify under DWF conditions with the F_o/M ratio of 0.18 and sludge age of 4.5 days. Figure E.58 shows that the adjusted model has significantly nitrified the influent to produce an effluent ammonia concentration of approximately 2 mg/l. Consequently, the F_o/M ratio did not require to be lowered further.

Using eqn. 19.1 the new synthetic reactor tank volume was calculated as **6702m³**. This is 3102m³ (+ 86%) greater than the actual volume at the WTP. This therefore shows the sensitivity of tank volume to the F_o/M ratio.

As with the BOD analysis the first test carried out was concerned with ascertaining the disruptive period which the worst period of rainfall within the January '70 profiles would give rise to i.e. the plant was placed under a combined flow rate of 600l/s for a duration of four days. From table 15.13 it can be seen that the associated ammonia concentration for flows of 600l/s was 10mg/l. The results are shown in figures E.59 to E.61.

It can be seen from figures E.59 to E.61 that the combined flows, as expected, cause a greater degree of disruption with respect to the nitrification process. A period of 60hrs was required before the plant could restore itself to normal functioning. This test was repeated, but with an influent concentration of 7mg/l

(corresponding to a flow rate of 700l/s). The disruptive period obtained for this test was compared with the disruptive period which was obtained in the previous analysis (influent concentration of 10mg/l). The results are shown in figures E.62 and E.63.

With the influent concentration of 7mg/l, it can be seen that during the period of combined loading the effluent ammonia concentrations are lower. This would obviously be expected due to the lower influent concentrations. However, the reduced substrate caused a greater reduction in the autotrophic biomass (fig. E.63). Consequently, when the dry weather flows re-established slightly worse effluent quality was observed and a greater disruptive period occurred. With the influent concentration of 10mg/l it was noted that the plant required 60hrs before it returned to its normal functioning, however with the influent concentration of 7mg/l this period was increased to 80hrs. It is therefore evident that the disruptive period can be expected to significantly increase with decreasing influent concentration.

19.10 Determination of Disruptive Periods

It was estimated from the above analysis that a critical active autotrophic biomass concentration of approximately 15mg/l was required before the plant could provide optimum treatment of the dry weather flows. Utilising the trend of the "biomass increase" which occurred subsequent to the wet weather period, as observed in fig E.63 an assessment was made of the expected disruptive period which would result with respect to various combined loading durations. This procedure is explained below:-

With reference to fig. E.63 it can be seen that an average daily increase in biomass concentration of approximately 3mg/l occurs for each day of subsequent dry weather flow. As it had been noted that a daily average concentration of approximately 15 mg/l would be required before the plant could restore itself to normal functioning, it was possible to calculate the length of time which would be required before the biomass would increase to this value e.g. it can be deduced from figure E.61 that if the plant were placed under a combined load of 600l/s for

a period of one day the concentration of bacteria within the reactor would be approximately 12mg/l at the end of the wet weather period. As the biomass increase by 3mg/l per day, a duration of only one day would be required before the biomass could increase to 15mg/l. Utilising this method the expected disruptive periods were calculated for the influent characteristics as shown below. Analysis was also carried out with respect to inflows of 800, 900 and 1000l/s. It was not considered necessary to show these figures as the same general trend, as shown in figure E.63 was observed. Table 19.9 below documents the results:-

Table 19.9 Disruptive Period - Summary Table

Flow	Combined Loading Duration of:-			
	1 day	2 days	3 days	4 days
600l/s	D.P. 1 day	D.P. 2 days	D.P. 3 days	D.P. 3 days
700l/s	D.P. 2 days	D.P. 3 days	D.P. 3 days	D.P. 4 days.
800l/s	D.P. 3 days	D.P. 4 days	D.P. 5 days	D.P. 5 days.
900l/s	D.P. 3 days	D.P. 4 days	D.P. 5 days	D.P. 6 days.
1000l/s	D.P. 3 days	D.P. 4 days	D.P. 5 days	D.P. 6 days.

where D.P. corresponds to Disruptive Period.

Combined loading durations of greater than four days were not tested as analysis of the historical rainfall profiles showed that the plant would not be subject to longer combined loading durations.

With reference to figures E.59 to E.63 it can be seen that although the plant does not return to its steady state for the durations defined in table 19.9 the plant does not discharge substandard effluent over this entire duration. Utilising the 600l/s test case as an example it can be seen that the time required for the plant to return to steady state after four days of combined loading was 3 days, however it can be seen that after only one day the effluent being produced from the plant was of a very reasonable quality. Consequently, the disruptive periods as defined in table 19.9 were the maximum durations which could be chosen. It could also be ascertained that the disruptive periods as defined above only apply if no additional rainfall occurs during the recovery period. Further tests were therefore

carried out to ascertain the affect which additional periods of combined loading would have upon the plant performance if they occurred whilst the plant was still in a disruptive state. The principal objective of these tests was to ascertain whether the subsequent flows would cause the same initial drop in biomass concentrations as would the initial event. From fig. E.63 it can be seen that this initial drop is quite significant (hour 24), and thus if a similar drop were to occur when the bacteria were not at such a high level the time required for the plant to restore itself to normal functioning would be substantially increased. Various inflow scenarios were used in the test however, the flow split prior to the reactor ensured that the reactor was being fed the same hydraulic loading for each of the test cases. Consequently, the same general trend was observed for each test. The results presented below therefore correspond to the 600l/s combined flow analysis as this represents most accurately the likely flow profile which the historical rainfall events produce. In order to carry out the test the plant was loaded with two sets of combined influent (600l/s), each with a duration of 4 days. An inter-event dry period of only one day was utilised to ensure the plant was in a disruptive state at the onset of the second set of combined loading. Figure 19.8 demonstrates the loading characteristics as described above. Fig.19.9 shows the consequential affect these loading conditions had upon the effluent ammonia.

Fig. 19.8 Bacteria Displacement Analysis - WTP Effluent – Flow

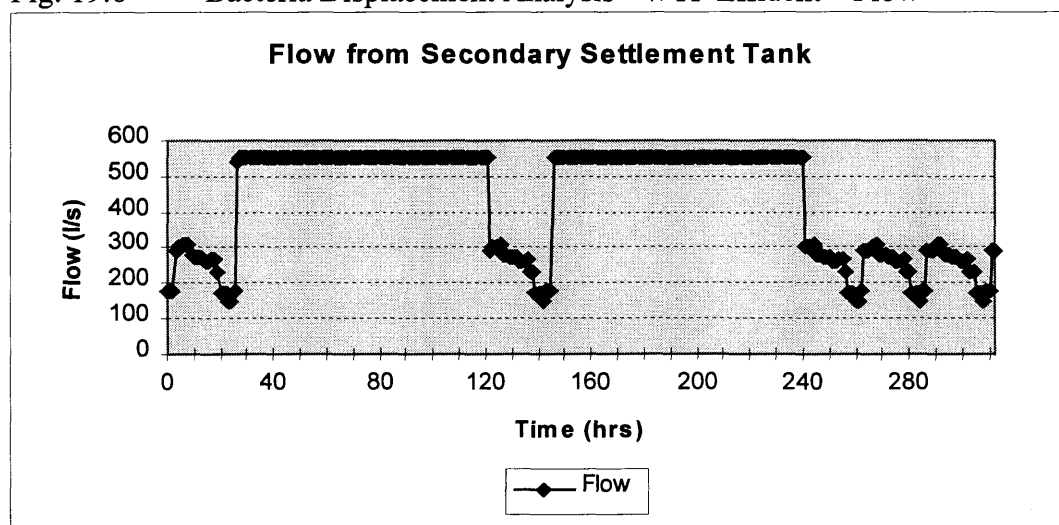


Fig. 19.9 Bacterial Displacement Analysis - WTP Effluent – Ammonia

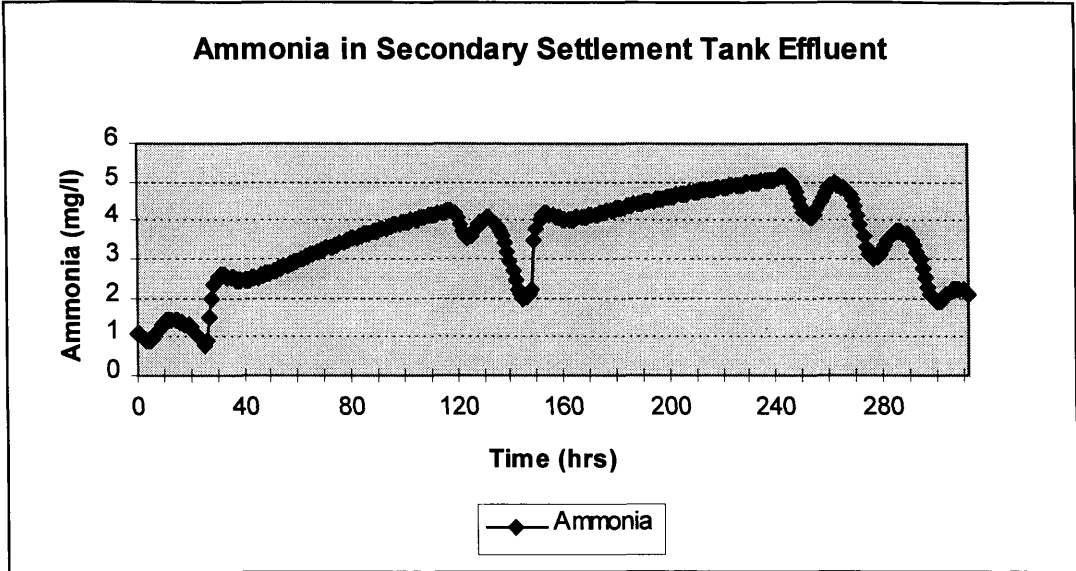
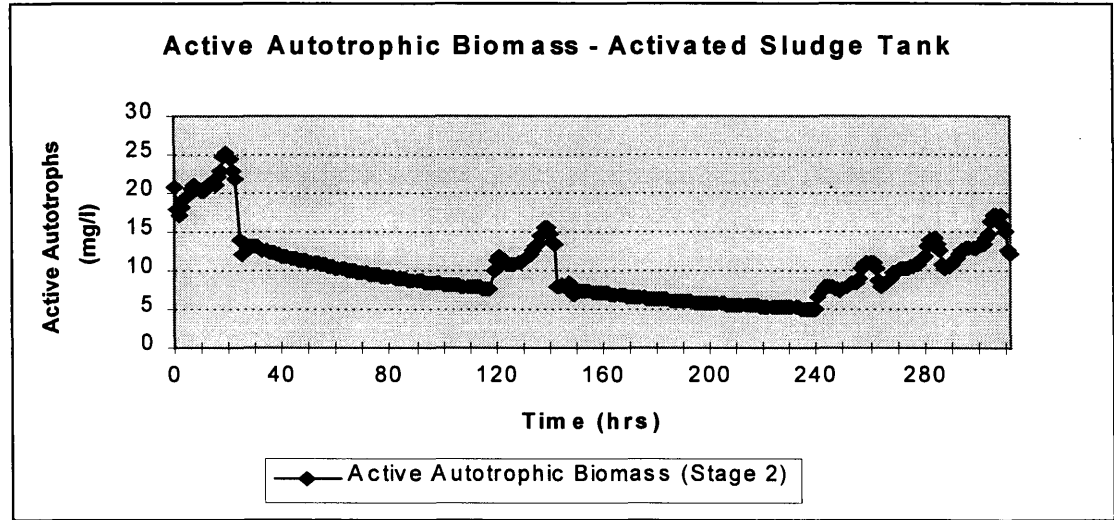


Fig. 19.10 Bacterial Displacement Analysis – Autotrophic Biomass in Activated Sludge Tank



With reference to fig. 19.10 it can be seen that the second period of combined loading (hour 148) does not cause as significant a drop in biomass as the first period (hour 24). This was unexpected and is attributable to the return sludge rate at the plant being sufficient to maintain a biomass concentration within the reactor of no lower than 7 mg/l, irrespective of the concentrations prior to the combined flows. This was confirmed via subsequent tests. Tests were also carried out to ascertain the drop in biomass concentrations which could be expected with an inter-event dry period of two and three days, however the same results were obtained (approximately 7 mg/l).

The only way in which biomass concentrations can be reduced lower than 7mg/l, at the Perth WTP, is therefore as a consequence of biomass decay due to limiting influent substrate. Utilising the above analysis, and the principal previously discussed, it would have been possible to produce a table which would show the disruptive periods which would result if additional periods of combined loading occurred whilst the plant was still in a disruptive state. This however, was not carried out as analysis of the historical rainfall files dictated that the subsequent events in the January '70 profiles were not of a sufficient magnitude (event 12/1/70:- vol. 1.5mm) to cause additional disruption at the plant. It was also noted from observations of the rainfall files that other periods of worst case rainfall would not subject the plant to combined flows for periods greater than four days. Consequently, the disruptive periods could be conservatively calculated using table 19.8 i.e. if the influent profiles were:- two days rain, one day (of inter event) dry weather, followed by two days of rain, then a conservative total emission analysis period could be derived from the disruptive period which four days of continuous combined loading would cause, plus an additional twenty four hours to account for the inter-event dry weather period. It can be seen from figures 19.9 & 19.10 that this approach would provide a conservative, but acceptable analysis period.

It became apparent during the actual total emission analysis that it would be sufficient to analyse total emissions simply over the worst period of combined loading i.e. no extended analysis period would be required. Consequently, sensitivity testing of a plant with respect to the disruptive period would therefore not be necessary in a normal total emission study. This is discussed in detail in the following chapter. This however, was not known at this stage and consequently, the analysis period with respect to the ammonia determinand was defined in terms of the plant disrupted period.

19.11 TEAP Summary

The objective of this chapter was to establish the TEAP for the Perth catchment. The first stage in the process was the screening of the historical rainfall data. This

was required to remove insignificant events. With respect to the Perth study such events were defined as those with a total rainfall depth of less than 1mm volume and with peak and average intensities less than 0.2mm per hour. The next stage in the process was the definition of the wettest summer and winter months, and from within these months, the wettest rainfall periods. In order to ascertain the WTP disruptive period which these events would cause a sample set was run through the SIMPOL model (hydraulic only). The Jan '70 profiles were used as the sample set. The objective of this analysis was to ascertain the average combined flow rate and duration which these events would give rise to. These characteristics were run through the WTP model to ascertain the resulting WTP disruptive period. Variations of these flow characteristics (different flow rates and durations) were then run through the WTP model to ascertain the likely disruptive period which different sets of rainfall events would produce. The objective of which was to provide a "look up" table which would show the WTP disruptive period for a range of combined loading rates and durations. This was considered to be important analysis as the worst case rainfall periods within the different months of the historic rainfall data would give rise to vastly different WTP influent characteristics. By ascertaining these influent characteristics via SIMPOL and utilising the "look-up" table, the WTP disruptive periods could be ascertained relatively quickly.

It was also discussed that a "rain-day" table of each month under analysis should be produced and utilised to ascertain the TEAP. The objective of the "rain-day" table is to ensure that the analysis is long enough to allow the WTP to return to steady state i.e. if a set of rainfall profiles cause a WTP disruptive period of five days subsequent to the end of the events, then the "rain-day" table can be used to ascertain if additional rainfall will occur within this period. If additional rainfall does occur then it is required to extend the TEAP until the plant is at steady state.

The WTP analysis as detailed in this chapter showed that for BOD, the TEAP required only to be as long as the combined loading period which would result from the initial period of rainfall i.e. extended analysis would not be required as

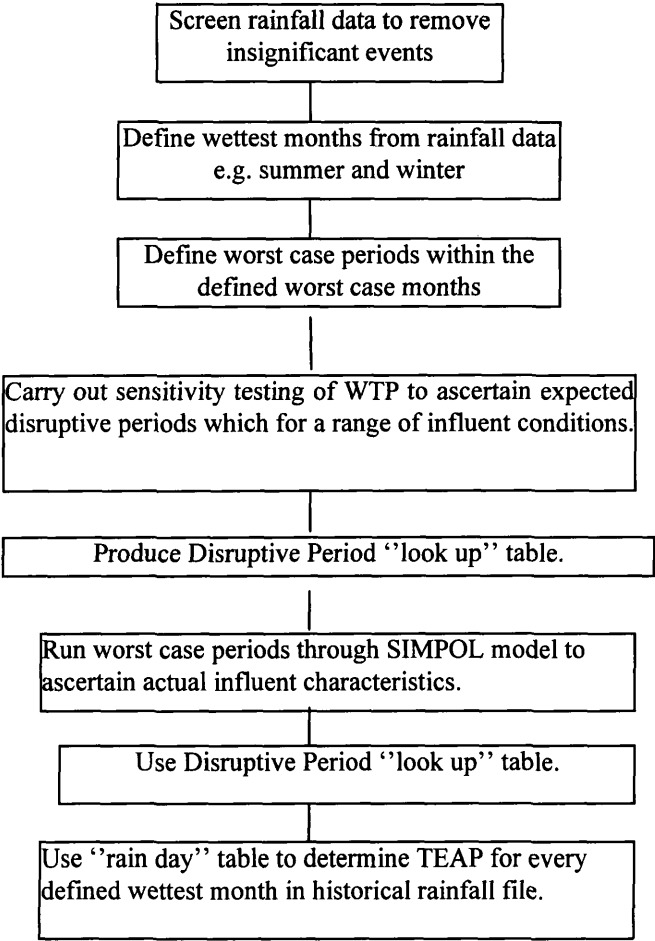
WTP disruption would not be expected to occur. This was due to two main reasons; the first was because the influent BOD concentrations were not dilute enough to cause the necessary reductions in biomass concentrations required to disrupt the BOD removal processes at the WTP, and secondly, because the rapid growth rates of the heterotrophic bacteria meant that if biomass concentrations were significantly reduced they would regenerate themselves quickly at the onset of dry weather flows. The only cases where a disruptive period was noted was when the average influent concentrations were equal to or less than 20mg/l. The analysis showed that when the influent concentrations were 20mg/l a maximum disruptive period of one day could result. The disruptive period was however observed to be independent of the combined loading period. Only when the influent concentrations were less than 20 mg/l was a greater disruptive period observed. The Perth data showed however that such low WTP influent concentrations would not be expected to occur.

It was observed that the worst case rainfall profiles in Jan '70 produced a WTP combined loading period of 4 days. Consequently, the BOD TEAP for the Jan '70 analysis was defined as 4 four days.

With respect to ammonia, it was observed that a continued disruptive period would be expected. This was a consequence of the slower growth rates of the autotrophic bacteria. Utilising the worst case rainfall profiles in Jan '70 it was observed that a three day period of WTP disruption would result. Consequently, the ammonia TEAP for the Jan'70 analysis was calculated as seven days.

Figure 19.11 summarises the general procedure for determining the Total Emission Analysis Period.

Figure 19.11 Method for Determining the Total Emission Analysis Period



Chapter Twenty

Total Emission Analysis (TEA) for Perth

20.1 Introduction

This chapter details the total emission analysis of the Perth system. In order to carry out the analysis the actual hydraulic outputs from SIMPOL were used to drive the WTP model. The quality characteristics as defined in table 15.13 were applied to these inflows depending upon the respective dilution which was provided. The derived flow weighted pollutant characteristics were also applied to the CSO spill data to produce the spill loads.

20.2 Total Emissions - BOD

Figure 20.1 shows the CSO spill loads with respect to BOD for the base system (no storage) and the system utilising the maximum (10,000m³) storage volume. Although the TEAP, with respect to BOD, was calculated as 96 hrs (4 days), the following graph(s) show 120 hrs of simulation data. The extra 24 hrs corresponds to one day of DWF prior to the onset of rainfall events. This has been included to allow the overall performance of the system to be observed. The benefit of this however, is seen with respect to the WTP performance as shown in figures 20.2, 20.3 and 20.4. As the beginning of the combined loading begins on 6/1/70 15:00, time zero on the following figures corresponds to 15:00 - 5/1/70.

Fig 20.1 CSO Spill Load - BOD (0m³ and 10,000m³ Storage)

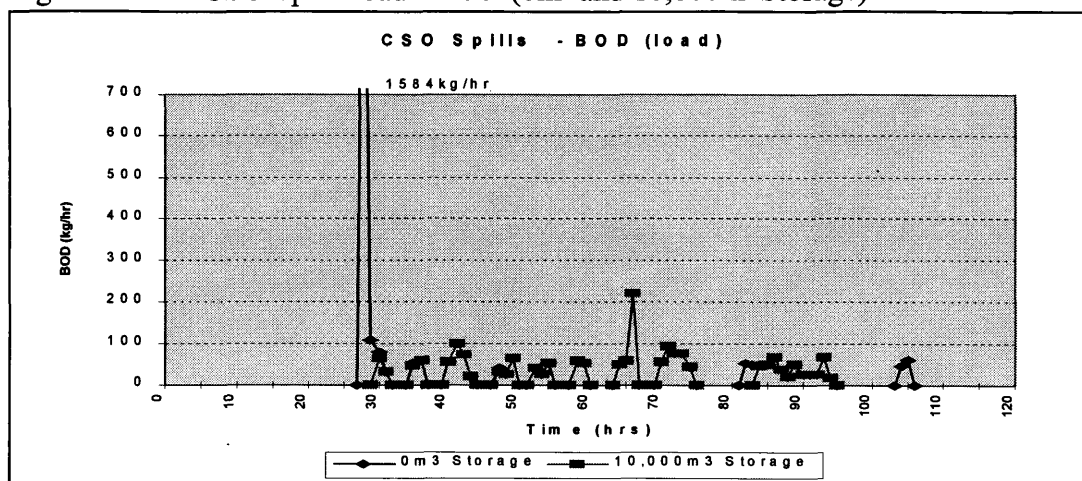


Figure 20.1 shows that subsequent to the initial period, the loads from the CSOs are essentially the same for the two scenarios (base system and 10,000m³ storage) with the exception of the initial period (27-30 hrs). During this period the detention tank is filling, and thus has captured the heavily polluted flush. CSO performance subsequent to this period is shown to be very similar, which is a consequence of the tank being full for the majority of the simulation. The pollutant loads from the respective systems are shown in table 20.1.

Table 20.1 CSO Spill Load – BOD (0m³ and 10,000m³ Storage)

BOD	0 m ³ Storage	10,000m ³ Storage	Difference in Load (kg)
Total Discharge Load (kg)	3,834	1,899	-1945

With reference to the loads shown in table 20.1 and from observation of figure 20.1 it can be seen that the vast majority of this difference is, as previously discussed, a consequence of the detention basin storing the flush. A small benefit is observed later in the event when the storage volume prevents additional spills however this benefit is significantly smaller as the flows being spilled are dilute.

20.3 WTP Performance

Figure 20.2 - 20.4 show the effect of the theoretical storage tank on WTP performance. It can be seen from figure 20.3 that the storage tank exerts both a positive and negative affect upon WTP performance. Between hours 75 to 85 the influent hydrograph is smoothed via the utilisation of storage, preventing BOD peaks in the WTP effluent. The negative aspect of the storage occurs between

hours 108 to 116. This period corresponds to the draining of the storage tank. The negative aspect however is not that the storage caused additional disruption at the plant but simply that the draining of the storage forces the WTP to treat a greater volume of sewage. The effect of this is that the plant discharges similar quality effluent, albeit for a longer duration. No additional disruption resulted because the additional flows did not exert a negative affect upon biomass concentration (which are responsible for degradation and thus WTP efficiency). Fig. 20.4 shows that the affect of storage upon biomass concentration was simply one of holding them at a lower concentration for a longer duration.

Due to the high growth rates of these bacteria, the biomass concentration returned to a high level quickly. From the sensitivity tests carried out in chapter nineteen, it was shown that a continued disruptive period resulted when influent concentrations were prolonged at 20 mg/l for a period of one day or greater. However, influent of such low concentration would not be expected to occur for periods as long as one day. With reference to the influent data utilised in the above analysis, the average daily concentrations were calculated as ranging from 75mg/l to 100mg/l. Consequently, no period of continued disruption was experienced at the plant. Nevertheless, even if the influent characteristics were of such a concentration to give rise to a disruptive period the utilisation of storage would not have exacerbated this problem relative to the original systems' performance, unless the detention tank was very large ($\gg 10,000\text{m}^3$). The reason being that the additional drain time resulting from $10,000\text{m}^3$ was calculated as approximately 8 hrs. All the sensitivity tests (chapter 19) showed that an additional loading period of 8 hrs (from any point) would not decrease biomass concentrations by an amount which would cause a relative reduction in WTP efficiency. Consequently, if disruption was experienced at the plant, the utilisation of storage would not have exacerbate the problem.

Fig 20.2 WTP Effluent - Flow (0m³ and 10,000m³ Storage)

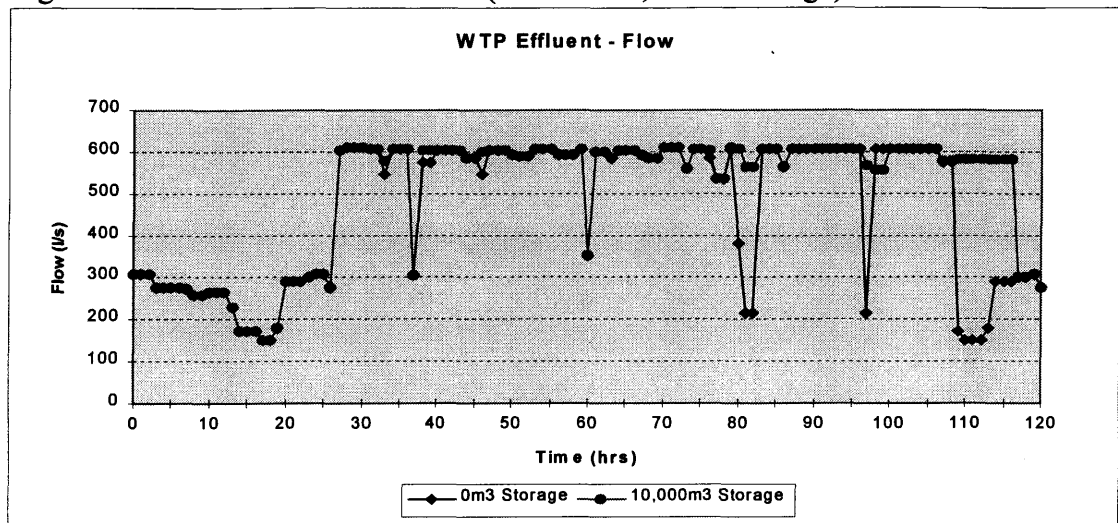


Fig 20.3 WTP Effluent - BOD (0m³ and 10,000m³ Storage)

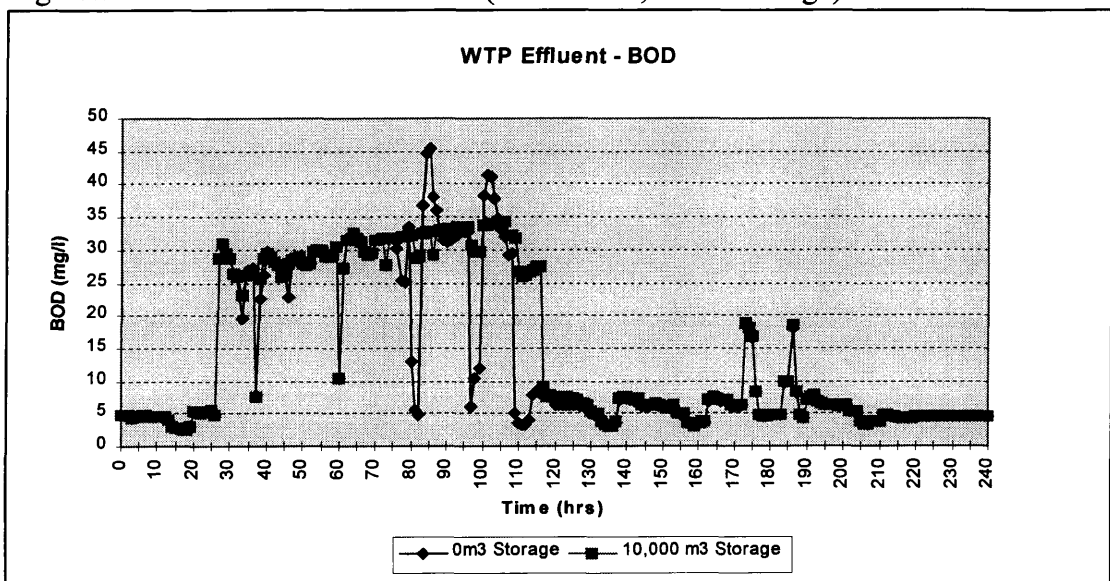
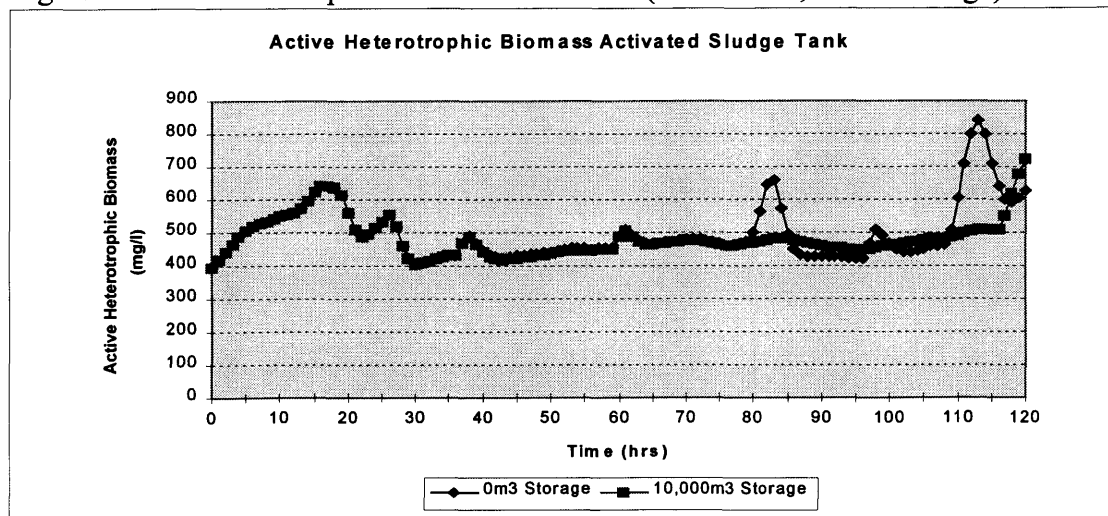


Fig 20.4 Heterotrophic Bacteria in Reactor (0m³ and 10,000m³ Storage)



The only case where disruption could be exacerbated would be where very dilute influent (10mg/l BOD) is fed to the plant. This therefore shows that in order for storage to cause a detrimental affect (relative to the unmodified system) the following two conditions are necessary:-

1. very dilute influent (<20mg/l BOD) which causes biomass concentrations to decrease with time.
2. very large storage volumes which increase the duration of combined loading to an extent which would causes a sufficient difference in biomass concentration relative to the reduction experienced in the original (no storage) system.

With reference to the Perth system, it can be concluded that a period of continued disruption would never occur (under normal operating conditions) as firstly the average BOD combined influent concentrations were substantially greater than 10mg/l, and secondly, even if these extremely low concentrations were to occur, the maximum storage volume would not be large enough to reduce the biomass to a level lower than that experienced in the unmodified system.

In terms of the general case, it is also very unlikely that periods of continued disruption would result. The reason for this is that the two aforementioned criteria are essentially conflicting i.e. very dilute sewage would only be expected from a system which does not generate large dry weather flows (low population) and is prone to significant infiltration. Such system would not require large volumes of storage. Consequently, the biomass would not be reduced relative to the reduction experienced in the unmodified system. Alternatively, large drainage catchments (which may require large storage volumes) should provide sufficient dry weather flows and thus sufficient influent substrate to prevent the influent BOD being diluted to such an extent (10mg/l) which would cause WTP disruption. Although this concentration of 10mg/l was based on the performance of the Perth WTP it is feasible to assume that this value can be applied on a more global scale. The reason being that all WTPs are governed by the same biological laws and therefore it can be hypothesised that similar results would be obtained

for different WTPs. This hypothesis is substantiated by **McQueen and James, (1997)**, who as previously discussed, stated that only very dilute influent would cause problems at WTP. Consequently, if the value required to cause disruption was found to deviate from 10mg/l from site to site the deviation would not be expected to be large. Verbal communications with **Dempsey, (1997)** substantiated the hypothesis that influent BOD, under normal conditions, would be unlikely to cause a continued period of disruption at a WTP. It was also discussed that the average influent BOD characteristics used in the UPM total emission assessment were 75mg/l. Consequently, it is not believed that storage would exert a negative affect upon WTP performance, with respect to BOD, for the Perth system and for the majority of normally operating WTPs. The only significant difference in WTP effluent quality which would be expected to result from storage would be the increase in load from the plant over the period of prolonged loading. With reference to the Perth analysis this period occurs between hours 108-116 (fig. 20.5).

Fig. 20.5 WTP Effluent Load – BOD (0m³ and 10,000m³ Storage)

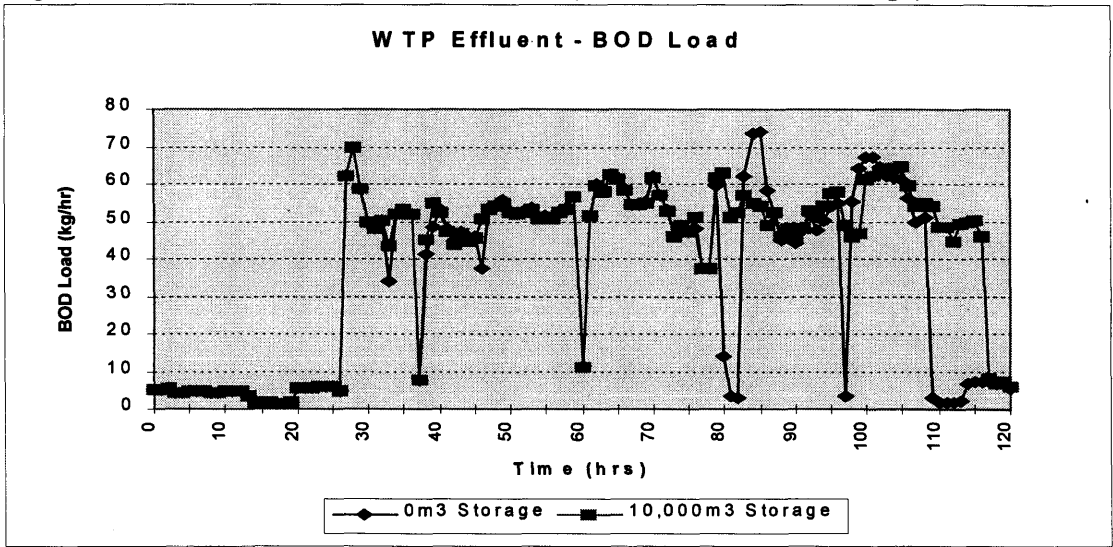


Table 20.2 WTP Effluent - BOD Load

BOD	0 m3 Storage	10,000m3 Storage	Difference in Load (kg)
Total Discharge Load (kg)	4288	4846	+558

Table 20.2 shows that the utilisation of storage has increased the BOD load by 558kg.

Table 20.3 shows the difference in total emissions which results as a consequence of utilising 10,000m³ of storage. A volumetric balance has also been provided to demonstrate that the results presented are meaningful.

Table 20.3 Volumetric Balance

Flow	0 m³ Storage	10,000m³ Storage	Difference (m³)
Discharge – CSOs (m3)	81,057	63,183	-17,874
Discharge – WTP (m3)	202,550	220,426	+17,876
Total Discharge (m3)	283,607	283,609	+2

A difference of 2 m³ is observed over the 120 hour duration, however this is attributable to rounding errors.

Table 20.4 Total Emissions - BOD

BOD	0 m³ Storage	10,000m³ Storage	Difference (kg)
Discharge - CSOs (kg)	3,834	1,899	-1935
Discharge - WTP (kg)	4,288	4,846	+558
Total Emissions (kg)	8,122	6,745	-1377

It can be seen from table 20.4 that the storage has provided an overall reduction in BOD discharges of 1,377kg.

20.4 Total Emission Analysis Summary (BOD)

The results of the analysis presented in this chapter highlight the effect which storage would have has on CSO and WTP emissions for the Perth system. It is clearly evident that although the storage increased the emissions from the WTP this was more than accounted for by the decrease in emissions from the CSOs (which principally resulted from the storage of the flush). Consequently, it can be concluded that even with large storage tanks an overall positive effect will be gained with respect to total emissions. This is a consequence of the dual effect of the storage tank preventing the flush being spilled directly to the receiving watercourse and the plant providing a certain degree of treatment to the wastewaters which are stored and passed forward. Without the detention basin these flows would be spilled directly to the receiving watercourse, without any purification. Consequently, this would result in a greater total emission load being discharged.

Since the plant provides a certain degree of treatment to the stored wastewaters it can also be concluded that even for cases where there is no flush (and thus the benefit of storage in terms of CSO spills would be significantly reduced) total emissions would still be reduced as the WTP treats the flows which are passed forward. Furthermore, due to the rapid growth rates of heterotrophic bacteria and the very low concentrations of influent substrate which are required to cause a problem, it can also be stated that other plants would not be expected to experience any period of continued disruption as a consequence of storage (this is true even for very large storage volumes). Consequently, a benefit in total emissions with respect to BOD would always be expected when in-sewer storage is utilised.

With reference to other total emission work (**FWR, 1994 & Durschlag et al, 1991**) it can be deduced that an overall benefit is gained in terms of total emissions for organic matter loadings for all ranges of storage volumes. This reinforces the conclusions derived here that storage will not cause a total emission problem with respect to BOD. Consequently, if analysis showed that a certain volume of storage did cause the receiving water quality standards to be breached it would logically follow that the scenario without storage would cause a greater breach since the total emission loads being discharged would be greater. It is possible however, that acute pollution problems can occur even although the storage has reduced the total emissions from the plant. This could result as a consequence of varying assimilative capacities within the receiving watercourse through out its diurnal cycle. This is a consequence of the photosynthesis and respiration processes of the in-stream vegetation (**Harremoes and Rauch, 1996**). These processes dictate that night time discharges are more detrimental to the receiving watercourses than day time discharges. As storage can cause the plant to discharge wet weather loads (of similar quality) through the night the lower assimilative capacities within the receiving watercourse during this period could potentially result in the receiving water quality standards being breached. This would require detailed receiving water quality modelling (and due to the sensitivity of this problem, more accurate sewer flow quality modelling data) to

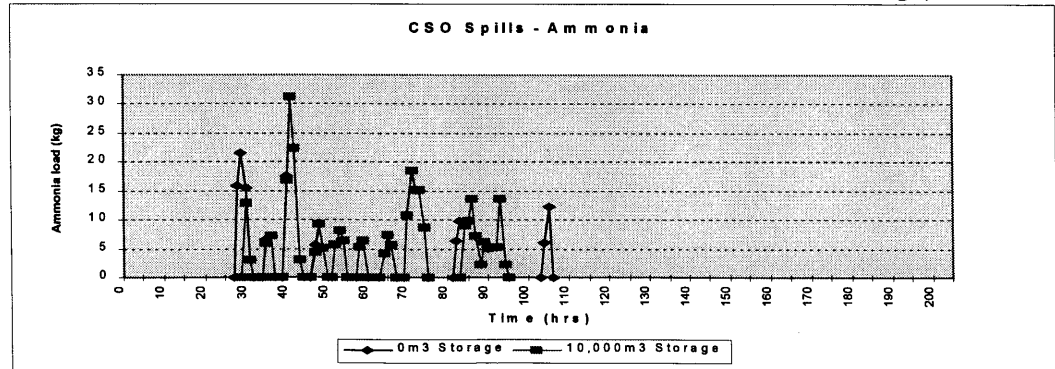
analyse this ‘potential’ problem properly. It is however believed to be a potential problem only in the more sensitive of receiving watercourse.

Nevertheless, detailed river modelling lies beyond the scope of this particular research project. As only simplistic sewer quality modelling tools can be used at present it would be recommended that significant advances in sewer flow quality predictive tools are made before this analysis is undertaken. Without this information it is believed that the main threat of storage with respect to total emissions (BOD) results solely from a ‘sludge blanket breakthrough’ from the final settlement tank. As previously discussed this problem could not be reproduced in the Perth system. This was expected as the problem of final settlement tank failure is documented as being related to shallow tanks which do not have the depth to accommodate the increase in sludge blanket depth which results under combined loading (**Harremoes and Rauch, 1996**). The method proposed in this thesis would however allow this problem to be thoroughly evaluated for systems which are, or may be subject to, final clarifier problems.

20.5 Total Emissions - Ammonia

The same analysis which was carried out for BOD was repeated with respect to ammonia. Due to the slower growth rates of the autotrophic bacteria the analysis period required to be extended by three days corresponding to a simulation end time of 00:00, 14/01/70. The performance of the CSOs for the two scenarios, 0m³ storage and 10,000m³ storage, are shown in figure 20.6:-

Fig.20.6 CSO Spill Load – Ammonia (0m³ and 10,000m³ Storage)



As the ammonia determinand does not produce a flush at the onset of the combined flows, the benefit of storage with respect to ammonia is significantly less than for BOD. The CSO spill load difference for the two scenarios are presented in table 20.5. It can be seen that only a 88kg difference in load is obtained over the 200 hour analysis period.

Table 20.5 CSO Spill - Ammonia

Ammonia	0 m³ Storage	10,000 m³ Storage	Difference in Load (kg)
Total Discharge Load (kg)	396	308	- 88

20.6 WTP Performance (Ammonia)

Figures 20.7-20.9 show the effect which the 10,000 m³ storage tank had upon the nitrification process. With reference to fig. 20.8 (hours 0 -24) it can be seen that normal effluent quality under dry weather conditions ranges from 1.5 to 2 mg/l. Due to the slower growth rates of the autotrophic bacteria, a period of approximately 90 hours is required before effluent of similar quality is discharged. This is true for both cases, with and without storage (i.e. the draining of the storage tank has not increased the disruptive period) although it can be observed that where storage has been utilised the effluent quality is slightly poorer (fig. 20.8 shows a peak difference of 1.5mg/l). This at first appears difficult to explain as the biomass concentrations, as show by fig. 20.9, at the end of the combined loading period are of a very similar concentration (approx. 6mg/l) however, for the scenarios without storage this concentration was increased to 14mg/l at the onset of the dry weather flows, whereas for the scenario with storage biomass concentrations increased to only 10mg/l. This caused an offset in biomass concentration for the remaining hours of the simulation, resulting in different effluent qualities being produced. The reason the bacteria do not increase to the same level can be explained with reference to figures 20.7. and 20.9. It is seen from fig. 20.9 that the large increases in biomass concentration occur at hours 80, 95, and 105. These increases correspond to the periods where the flow has dropped from 560l/s to approximately 200l/s (fig.20.7). Smaller peaks in biomass concentration can be observed at hours 35

and 60. These correspond to periods where the flow rate has dropped from 560l/s to 300l/s and 350l/s, respectively. It is therefore apparent that the increase in biomass concentration at the onset of dry weather flows is dependent upon the associated drop in flow rate which occurs. As dry weather flows are re-established, for the scenario without storage, at a period within the diurnal cycle conveying the lowest flow rate, the drop in flow is larger. This results in a larger increase in biomass concentration (relative to the scenario with storage, whose dry weather flows established at a period within the diurnal cycle conveying the largest flow). These two cases therefore represent the worst scenario (lowest and highest dry weather flow) producing the worst case difference in biomass concentration and thus effluent quality. This difference in effluent quality is therefore not a consequence of the storage causing greater disruption at the plant but simply as a consequence of the storage causing the dry weather to re-establish at different periods within the diurnal cycle. Consequently, if the wet weather loads for both simulations terminated at a period in the day of similar dry weather flow the effluent profiles from the plant would follow very similar patterns. This is demonstrated by figures 20.10, 20.11 and 20.12, which compare plant performance relative to storage volumes of 10,000m³ and 20,000m³. The storage volume of 20,000m³ increased the drain time by an additional 8hrs thus allowing the dry weather flows to re-establish at a period of similar flow (fig. 20.10).

Fig. 20.7 WTP Effluent – Flow (0m³ and 10,000m³ Storage)

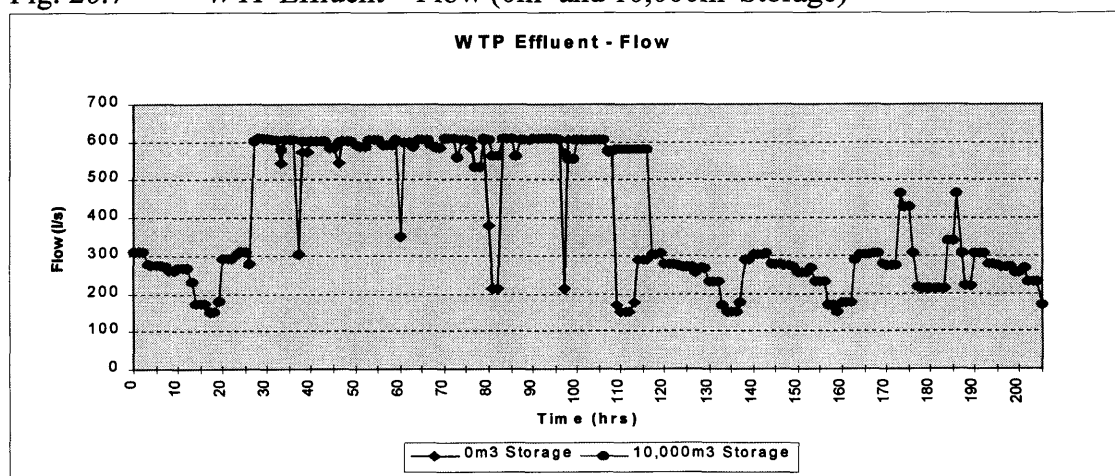


Fig. 20.8 WTP Effluent – Ammonia (0m^3 and $10,000\text{m}^3$ Storage)

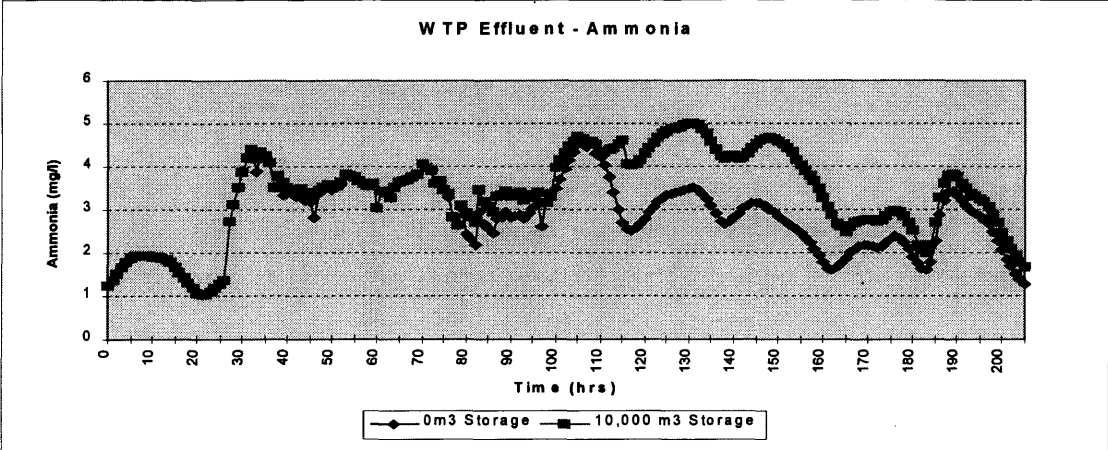


Fig. 20.9 Autotrophic Bacteria in Reactor (0m^3 and $10,000\text{m}^3$ Storage)

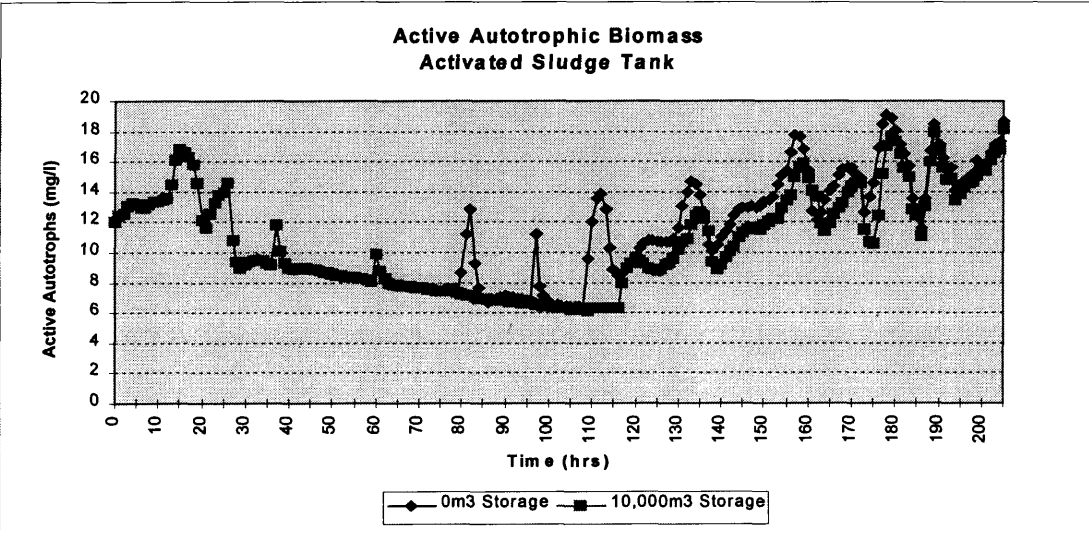


Fig. 20.10 WTP Effluent – Flow ($10,000\text{m}^3$ and $20,000\text{m}^3$ Storage)

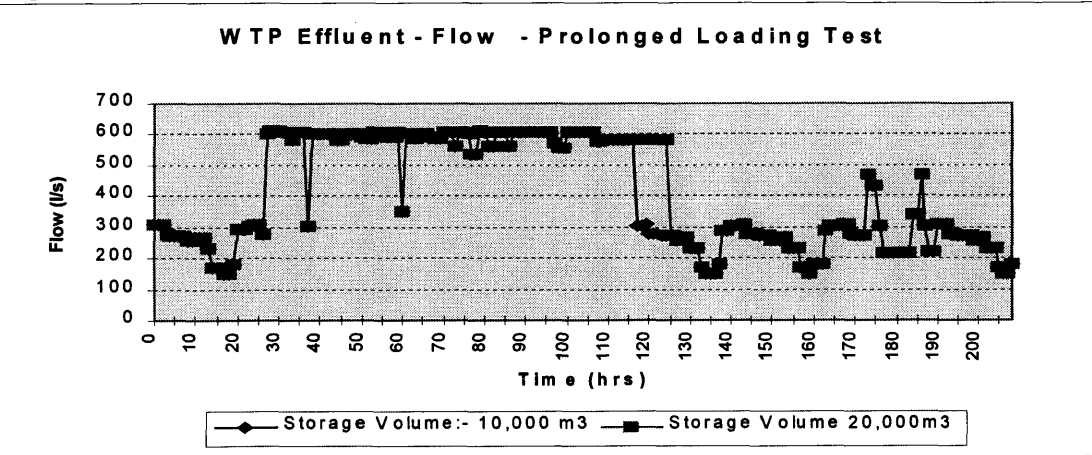


Figure 20.10 shows that the combined loading in both scenarios terminates at a period within the diurnal cycle of similar dry weather flow (hrs 116 and 124). This results in very similar trends in biomass concentration (fig 20.11) and consequently very similar effluent quality (fig. 20.12).

Fig. 20.11 Autotrophic Bacteria in Reactor (10,000m³ and 20,000m³ Storage)

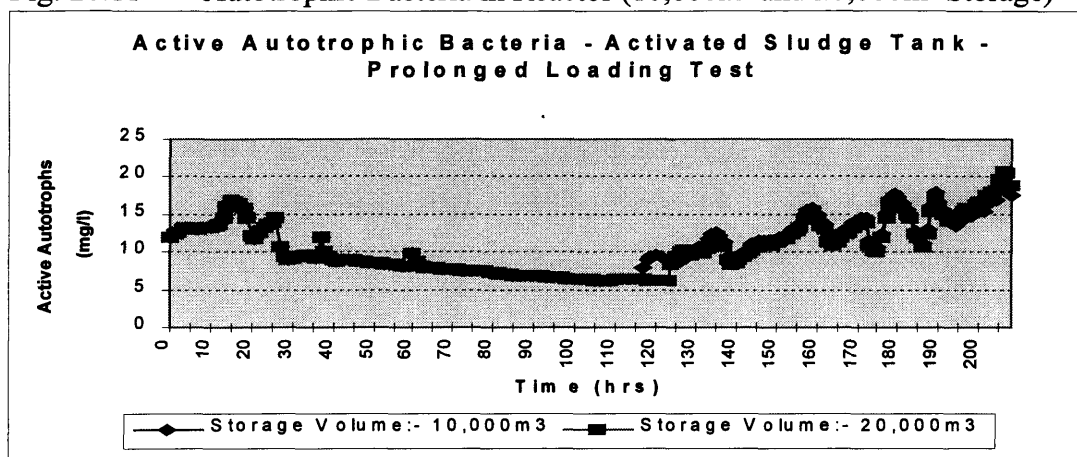
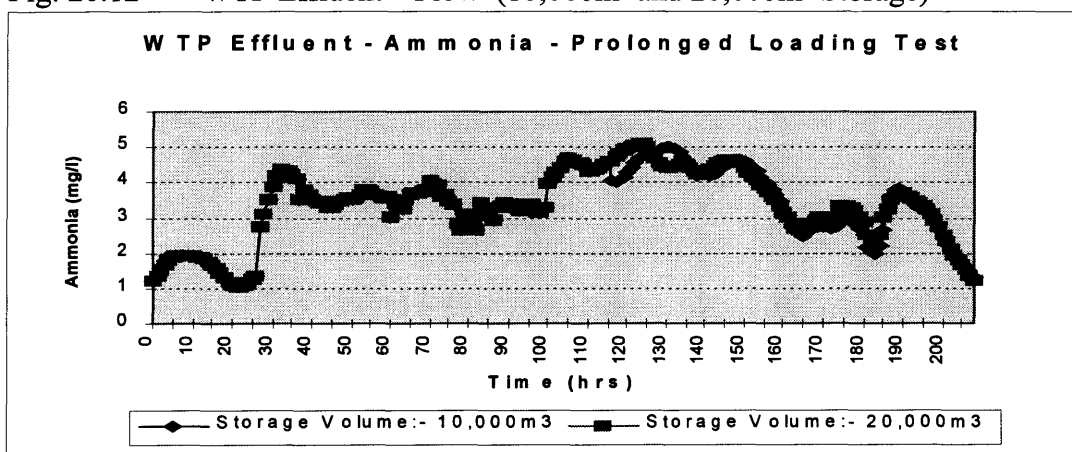


Fig. 20.12 WTP Effluent – Flow (10,000m³ and 20,000m³ Storage)



It can also be ascertained from figures 20.10-20.12 that even with a prolonged loading duration of 16 hrs, which corresponds to the drainage of 20,000m³ of storage, only very small reductions in biomass concentration are observed (relative to both the original system (no storage) and to the system utilising 10,000m³ of storage). Consequently, no varying degree of disruption is experienced at the plant. It is therefore evident that even very large volumes of storage will not exacerbate WTP performance. Consequently, the problem of storage in terms of ammonia total emissions is simply that the increased drain time can cause the dry weather flows to restore at different periods within the diurnal cycle. This problem can cause an offset in biomass concentration, which results in slightly poorer final effluent quality. With reference to the above analysis the worst case difference which resulted from this problem was very small (1.5 mg/l). It is was also noted that the offset bacteria problem is not

exacerbated by increasing storage volume as both small and large tanks can create exactly the same degree of disruption. The reason being that a drain time of only one hour could cause the dry weather flows to re-establish at a period within the diurnal cycle of a different magnitude. Fig. 20.13 and table 20.6 show the difference in WTP loads which resulted from the offset bacteria problem (which occurred whilst simulating with 0m³ and 10,000m³ of storage).

Fig. 20.13 WTP Effluent Load - Ammonia (0m³ and 10,000m³ Storage)

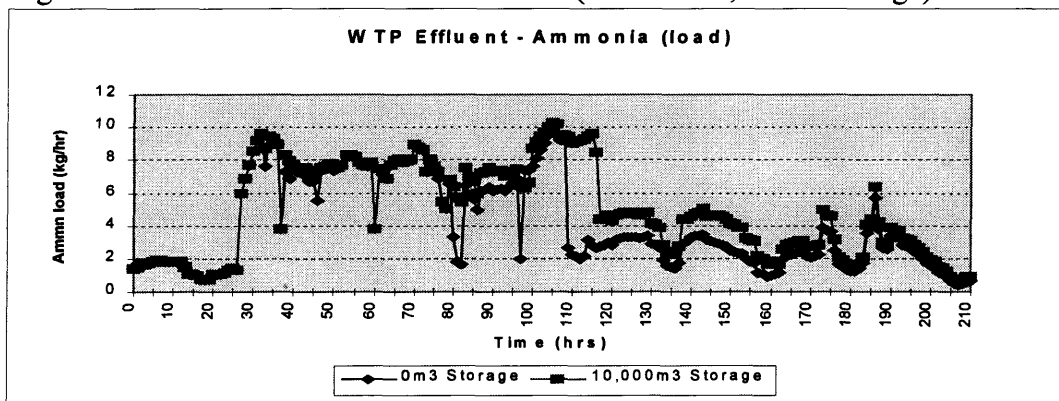


Table 20.6 WTP Effluent Load - Ammonia

Ammonia	0 m ³ Storage	10,000 m ³ Storage	Difference in Load (kg)
Total Discharge Load (kg)	865	1041	+ 176

Table 20.6 shows that WTP emissions were increased by 176kg as a consequence of treating greater volumes of sewage and the offset bacteria problem. The total emissions over the 205 hour analysis period are shown in table 20.7.

Table 20.7 Total Emissions - Ammonia

Ammonia	0 m ³ Storage	10,000m ³ Storage	Difference (kg)
Discharge - CSOs (kg)	396	308	-88
Discharge - WTP (kg)	865	1041	+176
Total Emissions (kg)	1,261	1,349	+88

Table 20.7 shows that an increase of 88kg occurs due to the bacteria differential within the WTP. It requires to be noted however that this increase could not be expected to exacerbate any potential acute pollution problems within the receiving watercourse as the increase occurs over the period of dry weather flows

which is subsequent to the period of combined loading. It can be clearly seen from fig.20.13 that the load discharged over this dry weather period is less than the load discharged over the combined loading period (hrs 24-116). Consequently, it is evident that the combined loading period is more critical in terms of acute pollution i.e. if the acute pollution standards were breached it would most likely be a consequence of the loads discharged over the wet weather period and not a consequence of the loads discharged over the period of continued disruption. The total emissions over the critical period (hrs 24-116) are shown in table 20.8.

Table 20.8 Total Emissions – Ammonia (Critical Period:- hrs 24-116)

Ammonia	0 m³ Storage	10,000m³ Storage	Difference (kg)
Discharge - CSOs (kg)	396	308	-88
Discharge - WTP (kg)	645	738	+93
Total Emissions (kg)	1,041	1,046	+5

The analysis of total emission over the critical period (hrs 24-118) shows only a 5 kg increase in ammonia total emissions (table 20.8). Although it would be expected that a reduction in total emissions would occur as a consequence of the plant treating, to a certain degree, the flows which were stored and passed forward, a marginal increase has been observed. This however can be explained with reference to fig. 20.8 which shows that between hours 80-95 the effluent ammonia concentrations for the system with storage are increased relative to the system without storage. This is a consequence of the storage tank causing different influent hydrographs (fig 20.7, hour 80). The effect of which being another offset biomass problem (*in which the respective bacteria concentrations for the storage/no storage scenarios differ simply due to the differences in the hydraulic loading conditions*). The resulting increased emissions over this period were observed to marginally counter the benefit which was gained by the plant treating the stored wastewaters.

It requires to be noted however that the general difference in effluent quality over hours 80-95 is only 0.5 mg/l. Although this serves to increase the total emissions over the analysis period it could not in itself be expected to exacerbate a potential

acute pollution problem. Furthermore, the increased emissions again, are not directly related to storage volume as increasing volumes of storage would neither alleviate or exacerbate this problem. The reason being that the problem is caused simply by the smoothing of the influent hydrograph. Any volume of storage which would smooth the hydrograph (small or large) would therefore cause the same problem. The analysis has shown that although storage would not exacerbate acute pollution problems it could not be expected to reduce the problem either (ammonia only). However, with reference to **Harremoes and Rauch, 1996** it requires to be noted that 'the distribution of the discharge load in space and time may exert a benefit upon receiving water quality' i.e. although no benefit, or only a marginal benefit, is observed with respect to the total emissions a benefit could be gained with respect to a localised pollution problem via the utilisation of storage at a CSO. The results do however support the argument that storage has limited positive effects for minimising acute pollution problems (**Harremoes and Rauch, 1996**). If an ammonia acute pollution problem were prevalent in a receiving watercourse improvements to the WTP, or the utilisation of Best Management Practises (BMPs) (**Pratt, C.J., 1995**), may therefore offer a better solution than in-sewer storage. As with the conclusions derived during the BOD analysis, it was however believed that storage could still cause an acute pollution problem as a consequence of the storage prolonging the plants combined effluent into a period of reduced assimilative capacity within the receiving watercourse. As previously discussed a detailed receiving water quality model would be required to complete this analysis.

20.7 Overall Total Emission Analysis Summary – BOD and Ammonia

The total emission analysis carried out for BOD has shown that:-

1. Storage smoothed the continuation flows to the WTP, preventing BOD peaks in the effluent. This was due to the TSS/BOD relationship which exists in the biological treatment process.

2. The observed influent BOD concentrations over the Total Emission Analysis Period were not dilute enough to cause a period of continued disruption at the WTP.
3. In order to cause a period of continued disruption the influent BOD concentrations would require to be maintained very low ($<10\text{mg/l}$) for a period much greater than 8 hrs relative to the no storage system.
4. As the maximum storage volume defined for the Perth system produced an extended loading period of 8 hrs, with an average influent BOD concentration in excess of 10mg/l , no period of continued disruption was observed for the Perth system. Consequently, no BOD total emission problem was observed.

The results obtained from the ammonia total emission analysis are summarised below:-

1. Ammonia does not exert a flush and thus the main benefit, which was observed with BOD (in terms of CSO emissions) was not observed for ammonia.
2. Prolonged periods of continued disruption were experienced subsequent to the period of combined loading as a consequence of the slow growth rates of the autotrophic bacteria. This was true for storage and no storage scenarios.
3. Storage volumes as large as $20,000\text{m}^3$ did not exacerbate the disruption experienced at the WTP. This was because the resulting increased loading period (which was 16hrs for $20,000\text{m}^3$) was not long enough to reduce the autotrophic bacteria to concentrations lower than those observed when no storage was utilised.
4. Increased total emissions were observed for ammonia as a consequence of in-sewer storage. The increases were observed to occur during the combined

loading period and during the period of continued disruption (subsequent to the combined loading period).

5. The increase in total emissions which occurred during the period of continued disruption resulted from the storage causing the dry weather flows to re-establish at a different period within the diurnal cycle (relative to the ‘no storage’ scenario). This proved detrimental with regard to the concentration of autotrophic biomass in the reactor. Consequently, poorer treatment of the re-established dry weather flows occurred as a result of storage. This problem was referred to as the offset bacteria problem.
6. The offset bacteria problem was shown not to be related to the size of storage volume as both large and small volumes held the potential to cause the same degree of WTP disruption.
7. The increased total emissions which occurred during the combined loading period were caused by the storage smoothing the hydraulic loading to the WTP. The smoothed flows prevented the increase in biomass concentrations which occurred in the ‘no-storage’ system. Although detrimental, the resulting differences in effluent quality were very small.
- 8 Ammonia total emissions were not reduced via the utilisation of storage.

The overall conclusions of the project are presented in chapter twenty one.

Chapter Twenty One

Conclusions

21.1 Introduction

Six aims and eight objectives were defined in the first chapter of the thesis. This chapter details the conclusion derived from the various aspects of the work and links them back to these original aims and objectives to demonstrate that the project has been successfully completed.

21.2 Modelling Conclusions

The first aim of the project was to develop and evaluate the limitations of computational simulation packages, to accurately represent the performance of the sewerage and treatment plant for the City of Perth. The various packages evaluated were:-

Sewer Hydraulics:-

Hydroworks-PM,

Sewer Flow Quality:-

MOSQUITO, Hydroworks-QM,

Wastewater Treatment Plant Performance:- STOAT and GPS-X.

The models were evaluated using data obtained from an extensive flow and quality data collection exercise (**objective one**) and the conclusions derived from the analysis of the respective packages are listed below:-

Hydroworks -PM

- Hydroworks-PM provided an accurate and reliable representation of the flows within the Perth sewerage system and was therefore maintained for use within the project. The good results were attributed to the full solution equations used in the model.

MOSQUITO

- The underlying hydraulic model (WALLRUS) was prone to instabilities.
- The output data from MOSQUITO proved insensitive to variations in the input pollutant data.
- This model was discarded for use within this project

Hydroworks-QM

- Reasonable quality modelling results were obtained for dry weather flows with the exception of ammonia. This problem could not be rectified.
- It was identified that infiltration was a significant component of the total dry weather flow in the Perth system (25%). Calibrating the base dry weather flow to the same level of accuracy whilst taking account of, and ignoring infiltration produced different sediment characteristics.
- Neither the ‘infiltration’ model or the ‘no infiltration’ model could reproduce an observed flush TSS profile for a main event within the calibration data.
- The problem was attributed to the model ignoring the deposited pipe sediment which pre-existed within the sewerage system.
- The model was observed to over-dilute the concentrations of pollutants in the combined flows subsequent to the first flush phenomena.
- Hydroworks –QM was discarded for use in this research project.

STOAT

Two version of STOAT were tested in this project; a prototype version of the software and the commercially available version.

- Reasonable results were obtained from the prototype version, however, the model was prone to over-predict final effluent BOD due to a simplifying hydrolysis assumption.

- The above problem was rectified via the release of the commercially available software.

GPS-X

- The GPS-X software provided more accurate results than the prototype version of the STOAT software and a similar degree of accuracy in comparison with the commercially available model.
- Both STOAT and GPS-X had problems representing the MLSS concentrations within the reactor. This was believed to be due to problems associated with representing a “non-controlled” system.
- GPS-X was utilised for the research programme for pragmatic rather than academic reasons.

SIMPOL was utilised in this research project, although no direct evaluation of the software was undertaken. This was because the modelling was carried out by NoSWA.

With respect to **objective two**, which was to assess the usefulness of the commercially available models, it was concluded that suitable tools existed to represent sewer hydraulics (Hydroworks –PM) and wastewater treatment plant performance (GPS-X). No commercial tools were available for the representation of sewer flow quality. A simplistic approach was therefore adopted which utilised hypothetical quality data derived from data collection. The first aim of the project (which was to develop and evaluate the limitations of existing software packages) was therefore met along with objectives 1 and 2. **Objective 4** (to utilise an existing STOAT model of the Perth WTP to represent BOD and ammonia) and **objective 5** (to produce final modelling tools suitable for the simulation of total emissions) were also met as a result of this evaluation.

21.3 Appropriate Computational Period

The second aim of the project was to develop a method for the selection of an appropriate computational simulation period required to investigate total emission whilst minimising computational time. The requirement to minimise computational time was based on the following:-

- Meaningful information with respect to intermittent discharge standards (acute pollution standards) required an analysis period much shorter than one year (as utilised in the German research).
- Discrete event simulation could not be used as the models would assume that storage was empty and that the WTP would be at steady state at the start of the simulation. These conditions could not be guaranteed and would therefore affect the loads discharged from the system (it was estimated for the Perth analysis that CSO spills would be under-predicted by 14,742m³ (over a four day period) if discrete event analysis had been utilised). As TSR events are analysed using discrete event simulation it was concluded that they would not be acceptable for the analysis of total emissions (**objective six**).

Based on the above criteria it was concluded that a meaningful analysis period for the analysis of total emissions would require to be long enough to ensure the WTP returned to steady state conditions. A method referred to as the Total Emission Analysis Period Method (TEAP) was therefore developed. This method was based on the following principles:-

- The total emission analysis period (TEAP) required to be of a reference duration from which the total emissions from all ranges of storage volumes can be compared,
- The TEAP must be long enough to ensure the WTP has returned to a steady state condition whilst accounting for the possibility of additional rainfall occurring within the WTP's disruptive period.

By limiting the analysis period to a duration which accounted for the WTP's return to steady state (in preference to using long term continuous simulation), the novel method ensured that an appropriate duration would be found which would provide meaningful information with respect to total emissions (in terms of acute pollution analysis). The method proved most critical for ammonia as the WTP disruption for this determinand was significantly longer than the disruption experienced for BOD for the same given rainfall profiles. Nevertheless, WTP analysis showed that the ammonia emissions discharged from the WTP during the period of continued disruption (subsequent to the combined loading period) were less than those discharged under the combined loading period. Consequently, it could have been concluded that the TEAP required only to be as long as the duration of combined loading which resulted from the selected worst case rainfall profiles (i.e. the TEAP would not require to be extended until the WTP returned to a complete steady state). Nevertheless, the TEAP still requires to be long enough to capture additional rainfall events which occur whilst the plant is disrupted. This is because the reduced bacteria concentrations within the reactor may not be sufficient to treat the re-established dry weather ammonia concentrations which are pushed out from the primary tanks by subsequent combined loading (**objective 7**).

As the method reduced the required computational time associated with continuous simulation, the second aim of the project was met via the development of this method.

21.4 Representative Rainfall

The third aim was closely linked to the development of the aforementioned TEAP method as it was concerned with the development of a method for the selection of a representative rainfall period for total emission analysis. This aim was met via the production of the "rain-day" method in which only the worst periods of rainfall within selected worst case wet months of a year were analysed. The justification for this method revolved around the following principle:-

- Meaningful acute pollution analysis requires a statistical expression like return period for which the concentration of dissolved oxygen (or a surrogate pollutant such as BOD) and/or ammonia should not be exceeded for a given length of time.

The “rain-day” method was therefore derived from the UPM intermittent standards as these complied with the aforementioned criterion. As these standards were based on statistical compliance it was only necessary to analyse worst case periods of rainfall from historical rainfall data to ascertain whether the standards were met.

21.5 Storage and WTP Performance

The fourth aim of the work was to further develop the knowledge about the problems which in-sewer storage of combined flows may cause in terms of disruption to WTP performance. Significant analysis was carried out in this area (chapter 19) and the conclusions derived from this work are listed below:-

BOD:-

- A significant portion of the MLSS were displaced to the final settlement tanks during the period of combined loading. This was a consequence of the increased hydraulic load.
- The controlled emptying of storage increased the duration of the increased hydraulic loading to the WTP.
- The diluted inflows reduced the concentration of active biomass in the reactor in accordance with traditional theory as proposed by Monod.
- Due to the high growth rates of heterotrophic bacteria no period of continued disruption was observed at the WTP subsequent to the combined loading period.
- Sensitivity testing of the WTP showed that a maximum disruptive period of one day could occur if the influent BOD concentrations were artificially reduced to 20mg/l. A longer period of continued disruption was shown to

be possible only when the influent BOD concentrations were reduced as low as 10mg/l.

- Such low influent BOD concentrations were not observed during the data collection exercise (the average daily influent concentrations were calculated to be between 75 and 100mg/l).
- Storage smoothed the influent hydrograph to the WTP, thus aiding the settlement performance of the final settlement (a balancing affect). A benefit was therefore observed with respect to final effluent TSS concentrations. An associated benefit was observed for BOD due to the TSS/BOD relationship.

The conclusions derived from the ammonia analysis are detailed below:-

- The hydraulic smoothing of the influent hydrograph had a detrimental affect with respect to the concentration of autotrophic bacteria within the reactor. This had an adverse affect on WTP emissions.
- Due to the low growth rates of the autotrophic bacteria a period of continued disruption was experienced at the WTP subsequent to the combined loading period.
- Storage, in general, did not exacerbate the disruption relative to the “no storage” system. This was because the prolonged combined loading period resulting from the controlled emptying of 20,000m³ of storage was not sufficient to reduce the autotrophic bacteria to a concentration lower than that experienced under the “no storage” scenario.
- A continued period of disruption was, nevertheless, observed under certain circumstances as a consequence of in-sewer storage. The problem was attributed to an “offset bacteria” problem in which the hydraulic loading from the storage caused the dry weather flows to re-establish at different periods within the diurnal cycle. When the combined loading period ended in a “low flow” period within the diurnal cycle a larger increase in autotrophic biomass concentration was observed in comparison to when the combined loading period ended within a “high flow” period. As the hydraulic

conditions affected the biomass concentrations different degrees of treatment were given to the re-established dry weather flows. This adversely affected the final effluent ammonia quality.

- The offset bacteria problem was not directly related to storage volume as both large and small volumes caused exactly the same degree of disruption.

21.6 Best Storage Volume

The fifth aim of the project was concerned with the development of a method to guide engineers through the process of defining the best storage volume for any particular catchment under consideration. The proposed method was introduced in chapter 16. The practical application of this method was applied to the Perth catchment (chapter 20). The conclusions derived from this analysis with respect to storage and BOD total emissions are detailed below:-

- In-sewer storage reduced the CSO spill load at the expense of increasing the discharged load from the WTP.
- The increased emission from the WTP were more than accounted for by the decreased emissions from the CSOs (storage of flush). This was because no period of continued disruption was experienced at the WTP.(the only cases where periods of continued disruption were observed was when influent BOD concentrations were artificially lowered to 20mg/l or less. However, such low influent conditions were not observed during the Perth data collection exercise).
- No volume of storage gave rise to a BOD total emission problem during the Perth total emission analysis.

It was therefore concluded that the best storage volume to prevent BOD acute pollution within a receiving watercourse would be the smallest volume which allowed compliance with the appropriate receiving watercourse standards.

The conclusions from the ammonia total emission analysis are detailed below:-

- Ammonia does not exert a flush and therefore the main benefit, which was observed for BOD (in terms of CSO emissions) was not observed for ammonia.
- Prolonged periods of continued disruption were experienced (subsequent to the period of combined loading) as a consequence of the slow growth rates of the autotrophic bacteria.
- The utilisation of storage volumes up to 20,000m³ did not exacerbate this problem (unless an offset bacteria problem was encountered). This was because the extended duration of combined loading resulting from the controlled emptying of 20,000m³ of storage was not sufficient to reduce the bacteria concentration within the reactor to a level lower than the concentration observed during the “no storage” analysis.
- Total emissions could be increased as a consequence of the offset bacteria problem as the reduced bacteria provided poorer treatment to the re-established dry weather flows.
- The increased loads discharged over the disruptive period were shown to be less than those discharged during the combined loading period (as the lower dry weather flows limited the load being discharged to the receiving watercourse). Consequently, these loads were not critical with respect to acute pollution.
- Storage was also observed to increase total emissions over the combined loading period (this period excludes the period of continued disruption). This was a consequence of the storage smoothing the influent hydrograph to the WTP. The smoothed hydrograph maintained the autotrophic biomass at lower levels than those observed in the “no storage” analysis, thereby detrimentally affecting WTP performance).
- It was therefore concluded that storage could increase ammonia total emission during and after the combined loading period, although the increased load would not be expected to exacerbate acute pollution problems.

The overall conclusion with respect to the project were therefore that all ranges of storage volumes would be expected to provide a benefit with respect to BOD total emissions. Consequently, the most appropriate storage volume would be the least volume which would allow compliance with the acute pollution standards. The novel TEAP method as proposed in this project should be used to ascertain this volume as discrete event simulation (using historical or time series rainfall) would under predict discharged loads due to the intrinsic assumption that storage tanks and WTP storm tanks are empty at the start of each event.

Where an ammonia acute pollution problem exists within a receiving watercourse, storage would not be expected to reduce the ammonia total emissions. It is possible however, that the distribution of the discharge load in space and time, as a consequence of in-sewer storage, may exert a benefit upon receiving water quality i.e. although no benefit, is observed with respect to the total emissions a benefit could be gained with respect to a localised pollution problem via the utilisation of storage at a CSO. Under these circumstances, the TEAP method could be used to size the appropriate storage volume. An overview of the TEAP method can be found at the end of this chapter. If storage is not expected to produce a localised receiving water quality benefit, alternative management techniques such as BMP's, or separation of combined systems could be investigated.

As **objective 8** was to utilise the computational tools, together with the analysis of disruptive periods to achieve the project aims, it can be concluded from the above that this objective was also met. With respect to **objective three**, which was to devise appropriate in-sewer storage volumes to alleviate intermittent discharges from the CSOs in Perth based around normal UPM procedures, it was concluded that storage would not be required due to the large assimilative capacity of the River Tay.

21.7 Two or Three DWF

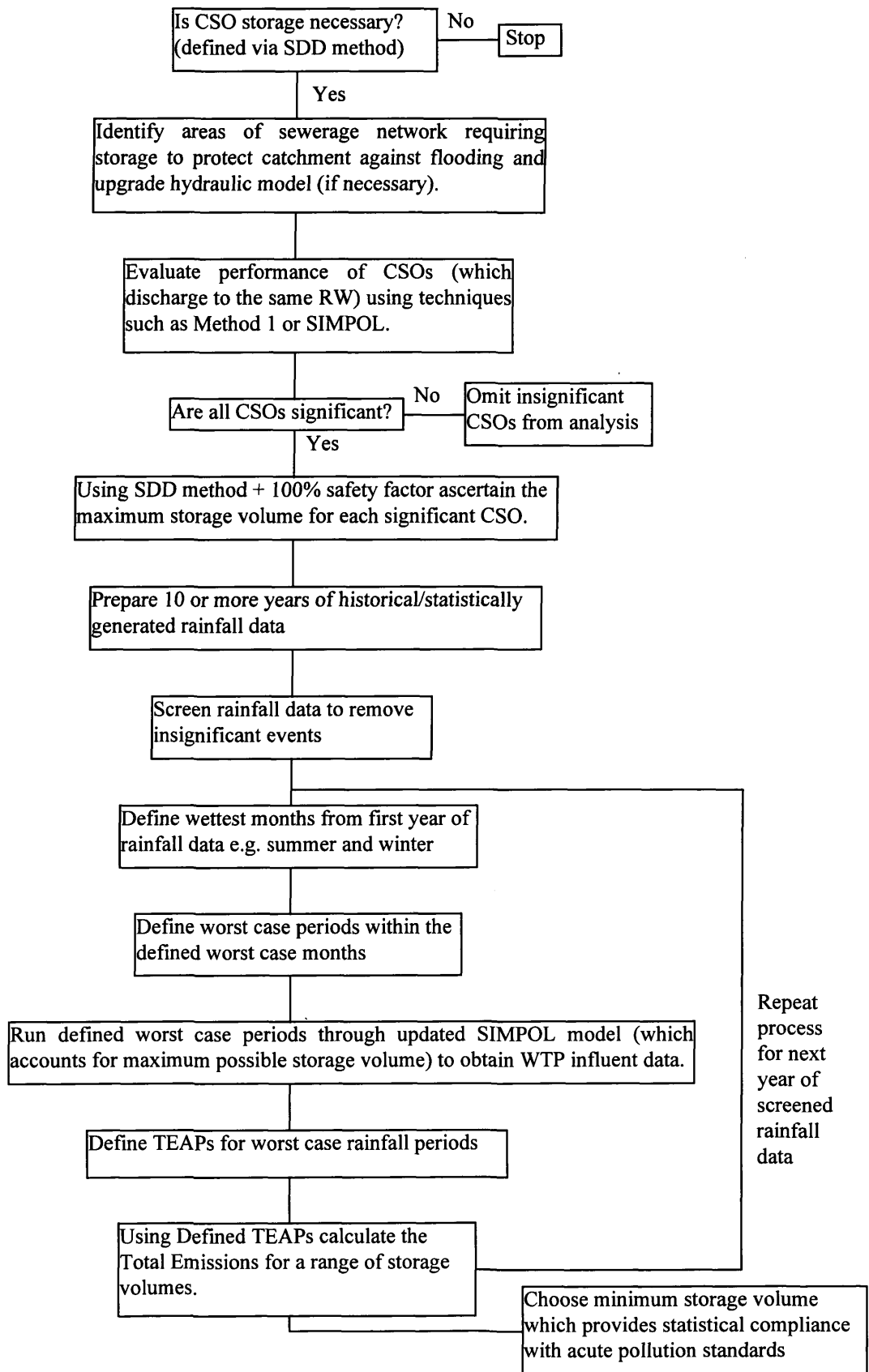
The final aim of the project was to assess whether full biological treatment of flows up to 2DWF would be preferable to the UK approach of treating flows of up to 3DWF. This analysis is detailed in Appendix F, however, it was concluded that:-

- Improvements to the nitrification process would be obtained by utilising a two dry weather flow to full to biological treatment limit. This was because the flow split reduced the hydraulic loading to the reactor, which helped maintain the autotrophic biomass concentrations at a high level.
- Better results were gained for BOD when the three dry weather flow to biological treatment limit was utilised. This was because the two DWF limit caused more of the influent BOD to bypass the treatment stage thereby increasing the final effluent BOD concentrations

The overall conclusions from this analysis were therefore that the biological treatment of flows up twice the DWF can not be considered better or worse than the treatment of three times the dry weather flow as no overall improvements were observed.

This chapter has demonstrated that all the aims and objectives of the research were met and that the project was successfully completed. Fig.21.1 overleaf provides a flow chart representation of the novel method which was produced via this project. The method can be considered as an improvement to current discrete event analysis methods for the sizing of storage volumes to alleviate intermittent discharge problems from sewerage systems comprising a WTP. A fuller description of the various stages of this method is detailed in chapters 16-20.

Figure 21.1 Summary of Total Emission Analysis Method



Chapter Twenty Two

Suggestions for Further Research

22.1 Introduction

Although the recent advances in computing technology have enabled the development of more complex tools and methods for evaluation of discharges to receiving watercourses, it is apparent that knowledge in certain important areas is still far from complete. Many areas would benefit from further research and the results from this could significantly aid practising engineers to produce cost effective, practical and more sustainable solutions to complex drainage catchment problems. The most salient areas requiring research are listed below:-

22.2 Sewer Flow Quality Modelling

As the sewer flow quality model drives all other models in an integrated study the importance of accurate quality modelling is paramount to the integrity of all 'real studies'. In order to improve sewer flow quality modelling predictions developments require to be made in the following areas:-

1. Sediment Transport Modelling:-

The cohesive structure of combined sewer sediments significantly influences the release of sediments. As the erosion of these sediments can release significant quantities of sediments which can be significantly organic, advances in the accuracy of prediction of sediment transport models require to be made.

A simplistic representation of the shear strength of the sediment bed is made within the available sediment transport models (one shear strength for a given sediment store). As the sediment bed in reality exhibits more complex behaviour than this, improvements in the predictions of sediment transport

equations could therefore be made by attempting to develop a relationship between the shear strength of the deposited bed with respect to depth, and with respect to time.

2. Pollutant Build Up Within the Sediment Bed:-

Available sewer flow quality models assume that the ‘user defined’ pollutants within the sediment bed remain constant with respect to time. These ‘defined’ concentrations however refer to the pollutants which were attached to the sediments at the time of the data collection exercise. If these sediments are eroded by a rainfall event, the associated pollutants will also be released into the flow. Consequently, if a sewer flow quality model is to have any long term integrity, research requires to be carried out to determine the likely concentrations which would remain in the sediment bed after erosion, and the rate at which these pollutants build up. A related area of interest would be to determine what levels these pollutants can build to.

3. Pollutant Release Mechanisms -

As the ‘blended’ laboratory technique provides unrealistic estimates of the pollutant release potential from sediments under storm conditions, research could be undertaken to provide a better means of ascertaining the pollutant releases from eroded sediment with respect to applied shear stresses. This work should be concerned with providing information in terms of potential pollutant releases from sediment under various flow characteristics. It would be a necessity to carry out this evaluation with consideration to the ADWP (which influences the concentrations of the pollutants within the sediment bed) for the reason discussed in 2. above.

22.3 Total Emission Analysis (Acute Pollution)

The method proposed in this project has provided significant information with respect to the technical appraisal of WTP performance in terms of varying storage volumes. The work however could be furthered if a detailed river model were used as important information could be provided with respect to the

potential problem of storage causing the WTP to discharge combined effluent throughout a period of reduced assimilative capacity within the watercourse. The information gained with respect to this analysis would help shed light on the significance of this potential problem.

22.4 Cost Benefit (Storage/ BMPs)

Investigations could also be carried out with a view to ascertaining the cost implications, and thus the feasibility of utilising a combination of BMPs and in-sewer storage volumes. It is believed that BMPs are a very useful alternative method of pollution prevention as they reduce the in sewer peak hydrograph, and thus the magnitude of pollutant release, and thus the discharged load. BMPs would also exert a positive affect upon WTP performance, via a reduction in combined loads. Consequently, cost benefit analysis would be useful in this area.

References

Arsov, R. (1993) Technological and Sanitary Aspects of the Reciprocal Impacts of the Combined Sewer, Wastewater Treatment Plant and Receiving Water. Proc. 6th International Conference on Urban Storm Drainage, Canada, Pg.1950-1957.

Ashley, R.M. (1993) Sediment Behaviour in Combined Sewers - Report on a programme of research for WRc. Wastewater Technology Centre, University of Abertay Dundee.

Ashley, R.M. and Crabtree R.W. (1992) Sediment Origins, Deposition and Build Up in Combined Sewer Systems. Wat. Sci. Tech. Vol. 25 (8).

Ashley, R.M., Wotherspoon, D.J.J., Coghlan, B.P. and McGregor, I. (1992a) The Erosion and Movement of Sediment and Associated Pollutants in Combined Sewers. Wat. Sci. Tech. Vol 25 (8) Pg.101-114.

Ashley, R.M., Wotherspoon, D.J.J., Goodison, M.J., McGregor, I. and Coghlan, B.P. (1992b) The Deposition and Erosion of Sediments in Sewers. Wat. Sci. Tech. Vol. 26 (5-6). Pg.1283-1293.

ATV (1992) Standards for the Dimensioning and Design of Stormwater Structures in Combined Sewers. German Pollution Control Guidelines.

Babtie, Shaw and Morton. (1993) Perth Flood Study Report. Babtie, Shaw and Morton, Consultancy Report. 95 Bothwell Street, Glasgow.

Bertrand-Krajewski, J.L., Lefebvre, M., Lefai B. and Audic, J.M. (1994) Flow and Pollutant Measurements in a Combined Sewer System to Operate a Wastewater Treatment Plant and its Storage Tank During Storm Events. The Sewer as a Physical, Chemical and Biological Reactor. Specialised International Conference, Aalborg, Denmark, May 16th-18th.

Blanc, D., Kellagher, R., Phan, L., Price, R. (1995) FLUPOL-MOSQUITO, simulations, critique et development. Proceedings NOVATECH '95. 2nd Int. Conf. on Innovative Technologies in Urban Storm Drainage. Lyon

Bryan, D. (1993) Evaluation of STOAT Model for Perth Sewage Treatment Works. Report No. UC 2152.

Bujon, G. (1988) Prevision des Debits et Flux Pollutants Transites par les Egouts par temps de Pluie. Le Modele FLUPOL. La Houille Blanche (1/1988) Pg.11-23.

Butler, D., Xiao, Y. Karunaratne S.H.P.G. and Thedchanamoorthy, S. (1994) The Gully Pot as a Physical, Chemical, and Biological Reactor. The Sewer as a Physical, Chemical and Biological Reactor. Specialised International Conference, Aalborg, Denmark, May 16th-18th.

Capodaglio, G. A. (1994) Integral Control Requirements for Sewerage Systems. Water Quality International '94. IAWQ 17th Bennial International Conference, Budapest, Hungary, 24th - 29th July.

Cordolia-Malina, J.T., Hudgkins, R.R. and Silverstone, P.L. (1979) The Gravity Clarifier as a Stratified Flow Phenomenon. The Canadian Journal of Chemical Engineering Vol 57. June, Pg.249-254.

Crabtree, R.W., Dempsey, P., Becker, M., Gent, R. J. Simpson, K. and Bryan, D. (1993) UPM Applications Methodology Final Report. Foundations for Water Research, Report No. FR 0384.

Dempsey, P. (1997) Verbal Communications Concerning Minimum WTP Influent Concentrations During Periods of Combined Loading.

De Vantier, B.A. and Laroc, B.C. (1987) Modelling Sediment Induced Density Current in Sedimentation Basins. Journal Hydraulic Engineering Vol. 113, (1).

Dudley, J. (1996) Verbal Communications Concerning Hydrolysis Assumptions within STOAT.

Dunn-Flores, A. (1998) Verbal Communications with North West Water's Wastewater Treatment Plant Expert Concerning the Utilisation of Equalisation Basins.

Durschlag, A. (1989) Bemessung von Mischwasserspeichern in Nachweisverfahren unter Berücksichtigung der Gesamtemission von Mischwasserentlastungen und Klaranlagenablauf, Schriftenreihe für Städtenwasserung und Gewässerschutz, Hannover.

Durschlag, A., Hartel, I., Hartwig, P., Kaselow, M., Kollatsch, D., Otterpohl, R. and Schwenter G. (1991) Total Emissions from Combined Sewer Overflows and Wastewater Treatment Plants - Report of the German Study Group. European Water Pollution Control Vol. 1 (6) Pg.13-23.

Durschlag, A., Hartel, I., Hartwig, P., Kaselow, M., Kollatsch, D., Otterpohl, R. and Schwenter G. (1992) Joint Considerations of Combined Sewerage and Wastewater Treatment Plants. Wat. Sci. Tech. Vol. 26 (5/6) Pg.1125-1134.

Field, R. and Turkeltaub, R. (1981) Urban Runoff Receiving Water Impacts: Program Overview. Journal of Env. Eng. Vol. 107 Pg.83-100.

Fraser, A.G. (1995) Pipe Roughness Assessment of the Perth Sewerage System. Report for North of Scotland Water Authority.

FWR (1994) Urban Pollution Management Manual: A Planning Guide for the Management of Urban Wastewater Discharges During Wet Weather. Foundation for Water Research Report FR/CL 0002.

Garsdal H., Mark, O. Dorge J., Jepsen S.E. (1995) MOUSETRAP: Modelling of Water Quality Processes and the Interaction of Sediments in Sewers. Wat Sci and Tech. Vol. 26 (5/6) Pg.1125-1134.

Gent, R. (1994) Verbal Communications Concerning COD Modelling Within MOSQUITO.

Gent, R., Crabtree, R.W. and Eperson, M. (1994) Improved MOSQUITO Applications Procedures. FWR Report No. FR 0443.

Gent, R., Crabtree R.W. and Ashley R.M. (1995) Sewer Sediments Research in the UK: Progress to date and Future Requirements. Int. Conf. on Sewer Solids: Characteristics, Movements, Effects and Control. Dundee, September 1995. Pg. 6-14.

GPS-X Technical Reference (1994) GPS-X Operational Guidance Manual. Produced by Hydromantis Inc.

GPS-X User's Guide (1994) GPS-X Operational Guidance Manual. Produced by Hydromantis Inc.

Guderain, J., Durschlag, A. and Bever, J. (1997) Total Emissions from Biological Reactors in Wastewater Treatment Plants and Combined Sewer Overflows. The Sewer as a Physical, Chemical and Biological Reactor. 2nd Int. Conf. 25 - 28th May, Aalborg.

Harremoes, P. (1989) Overflow Quantity, Quality, and Receiving Water Impact. Urban Discharges and Receiving Water Quality Impacts, (Adv. Wat. Pollution Control No.7) Pergamon Press, Oxford, U.K. Pg.9-16.

Harremoes, P., Capodaglio A.G., Hellstrom B.G., Henze, M., Jensen, K.N., Lynggard-Jensen, A., Otterpohl, R. and Soeberg, H. (1993) Wastewater Treatment Plants Under Transient Loading - Performance, Modelling and Control. Wat. Sci. Tech. Vol. 27 (12) Pg.71-115.

Harremoes, P. and Rauch, W. (1996) Integrated Design and Analysis of Drainage Systems Including Sewers, Treatment Plant and Receiving Waters. Journal of Hydraulics Research,. Vol. 34, 1996. No.6. Pg.815-826.

Henderson, R. J. and Dempsey, P. (1990) Estimating the Frequency of Operational of Storm Outfalls and Overflows - Method 1. Coastal Sewerage Research - Report No. 5. Contract No.6947. March 1990

Henze, M. (1987) Storm Water Handling in Waste Water Treatment Plants. EWPCA - Symposium '87. Munich, May, Pg 173-194. Gesellschaft zur Forderung der Abwassertechnik, e.V., St Augustin.

Henze, M., Grady, C.P.L., Gujer, W., Marais, G.R. and Matsuo, T. (1986) IAWQ Task Group on Mathematical Modelling for Design and Operation of Biological Wastewater Treatment. Activated Sludge Model No.1 Report Completed July 1986.

Howard, C.D.D. (1993) Sewer and Treatment Plant Operations Control for Receiving Water Protection. Proc. 6th International Conference on Urban Storm Drainage, Canada. Pg.1933 – 1937.

Hutchison A. McG. (1995) The Importance of Hydrogen Sulphide Generation in Sewerage Systems. B.Eng (Hons) Project. School of Construction and Environment, University of Abertay Dundee.

Jack, A.G. (1995) Hydraulic Evaluation of the Perth Sewerage System - Case Project Report for the North of Scotland Water Authority.

Jack, A.G. and Petrie, M.M (1995) The Capabilities and Limitations of Modelling in Holistic Management Practise. European Modellers Group. IMUG 95. The Pembroke Hotel, Blackpool 22nd November 1995.

Jack, A.G., Petrie M.M. and Ashley R.M. (1995) The Diversity of Sewer Sediments and the Consequences for Sewer Flow Quality Modelling. Int. Conf. on Sewer Solids: Characteristics, Movements, Effects and Control. Dundee, September 1995. Wat. Sci. Tech. Vol.33 No.9 Pg.207-214 1996.

Kappler, J. and Gujer, W. (1993) Influence of Operational Problems in Wastewater Treatment plants on the Interaction Between Sewers, Treatment Plants and Receiving Water. Wat. Sci. Tech. Vol 27 (12) Pg.199-203.

Kleijwegt R. A, Veldkamp R. G. and Nalluri C. (1990) Sediments in Sewers: Initiation of Transport. Wat Sci. Tech. 22 (10/11), Pg.239-246.

Kollatsch, D. (1993) Comprehensive Planning of Urban Drainage and Waste Water Treatment Plants. Wat. Sci. Tech. Vol. 27 (12) Pg.205-208.

Larsen, P. (1977) On the Hydraulics of Rectangular Settling Basins, Experimental and Theoretical Studies. Department of Water Resource Engineering, Lund Institute of Technology, University of Lund Sweden.

Lijklema, L. (1992) Integrated Management of Urban Waters - A Preface. Wat. Sci. Tech. Vol. 27 (12) Pg.vii -x.

Lijklema, L., Tyson J.M. and Lesouef A. (1993) Interactions Between Sewers, Treatment Plants and Receiving Waters in Urban Areas: A Summary of the Interuba '92 Workshop Conclusions. Wat. Sci. Tech. Vol 27 (12) Pg.1-29.

Lindholm O.G. (1976) Pollution Analysis of Combined Sewer Systems. Journal of Env. Eng., Vol. 102, Pg.301-312.

Londong, J. (1994) Consequences of the Behaviour of Activated Sludge Plants with Combined Sewage Inflows. Water Quality International '94. IAWQ 17th Bennial International Conference, Budapest, Hungary, 24th - 29th July. Pg.183-191.

Lumley, D.J. and Balmer, P. (1987) Full Scale Investigations of Secondary Settler Dynamics. Proc. 10th Symposium on Wastewater Treatment, Montreal. Pg. 21- 33.

Lumley, D.J. and Balmer, P. (1990) Solids Transport in Rectangular Secondary Settlers. Water Supply Vol 8. Pg. 123-132.

McGregor, I. (1995) Integrated Management of Sewerage Systems and Wastewater Treatment Plants. Perth Sewage and Sewer Data Report. Wastewater Technology Centre, University of Abertay Dundee.

McQueen, A. and James, G. (1997) Verbal Communications Concerning the Bacteria 'Die off' Phenomenon Experienced at WTPs.

Metcalf and Eddy. (1991) Treatment, Disposal and Reuse. Third Edition. Publishers: McGraw Hill International Editions. (Civil Engineering Series).

MOSQUITO User Manual (1993) WALLRUS - MOSQUITO Module Version 1.5 User Documentation First Edition 1st July HR Wallingford.

NRA (1993) Guidelines for AMP2. Periodic Review, Version 2. Effluent Quality Approved by the Quadipartite Meeting 14/12/93.

Otterpohl, R. (1993) Regenwasserbehandlung in kommunalen Klaranlagen mit Mischkanalisation, abwassertechnik, Heft 4/93.

Otterpohl, R., Freund, M., Sanz, J.P. and Durschlag, A. (1994) Joint Consideration of Sewerage System and Wastewater Treatment Plant. Water Quality International '94. IAWQ 17th Biennial International Conference, Budapest, Hungary, 24th - 29th July.

Parker, D.S. (1983) Assessment of Secondary Clarification Design Concepts. Journal WPCF 55. No 2 Pg.173-182.

Petrie, M.M. (1994) Perth Catchment Management Plan. Perth WALLRUS Model. Tayside Regional Council Internal Report January 1994 S3/144/B

Petrie, M.M. (1997) The Optimum Management of Wastewater Systems:- A Strategy for the City of Perth. Ph.D. Thesis. University of Abertay Dundee.

Phan, L., Herremans L., Delaplace, D. and Blanc, D. (1994) Modelling Urban Stormwater Pollution: Comparisons between British (MOSQUITO) and French (FLUPOL) Approaches. Hydroinformatics '94, Verwey, Minns, Babovic & Maksimovic (eds) Balkema, Rotterdam. Pg.443-450.

Pratt, C.J. (1995) A Review of Source Control of Urban Stormwater Runoff. J. Chartered Institute of Water and Environmental Management, 1995, Vol. 9, Pg. 132-139.

Sanderson, S. (1996) Verbal communication concerning Ammonia modelling within Hydroworks -QM.

Schutz, M. (1998) Integrated Modelling of Sewerage Systems and Wastewater Treatment Plants. Thesis in Preparation. Imperial College, University of London.

SDD (1977) Working Party on Storm Sewage (Scotland), Storm Sewage Separation and Disposal. HMSO Edinburgh.

Sedarity, A. (1996) Verbal Communications Concerning WTP Modelling.

Stotz, G. and Krauth, K.H. (1986) Detention Basins and their Capabilities for the Retention of Pollutants. 1st Wageningen Conference on Urban Storm Water Quality and Effects Upon Receiving Waters, CHO-TNO. October.

Threlfall, J. and Cowpertwait, P. (1996) Rainfall for Urban Storm Drainage Modelling. Seventh International Conference on Urban Storm Drainage. Hannover, Germany. Vol. 1. Pg.1-6.

WaPUG Code of Practise for the Hydraulic Modelling of Sewers (1998) Wastewater Planners User Group Code of Practise for the Hydraulic Modelling of Sewers Systems, November 1998.

Wotherspoon, D.J.J.W. (1995) Verbal Communications Concerning Sewer Flow Quality.

Wotherspoon, D.J.J. and Ashley, R.M (1992) Rheological Measurement of the Yield Strength of Combined Sewer Sediment Deposits. Wat. Sci. Tech. 25 (8) Pg165-169.

Wotherspoon, D.J.J., Petrie, M.M., Stenhouse, G. and Crabtree, R. (1996) Applications of STORMPAC and SIMPOL. WaPUG User Group. Spring Meeting, Birmingham.

Verbanck, M. (1994) Capturing and Releasing Settleable Solids - The Significance of Dense Under-currents in Combined Sewers Systems. The Sewer as a Physical, Chemical and Biological Reactor. Special International Conference, Denmark, May 16th -18th.

Appendix A

Plates

List of Plates

Plate 1	Flap Valve Overflow (Craigie)	A3
Plate 2	Overflow Pipes (Bridgend)	A3
Plate 3	Single Sided Weir Overflow (Bridgend)	A4
Plate 4	High Flows in Perth Sewerage System	A4
Plate 5	Low Volumetric River Flows	A5
Plate 6	Aesthetic Pollution	A5
Plate 7	South Inch Overflow Operation	A6
Plate 8	Sludge Injection – Friarton Pumping Station	A6
Plate 9	Pre-Treatment at Sleepless Inch WTP	A7
Plate 10	Primary Settlement at Sleepless Inch WTP	A7
Plate 11	3 Lanes of Settled Sewage	A8
Plate 12	Activated Sludge Plant	A8
Plate 13	Secondary Settlement Unit – Sleepless Inch	A9



Plate 1

Flap Valve
Overflow (Craigie)

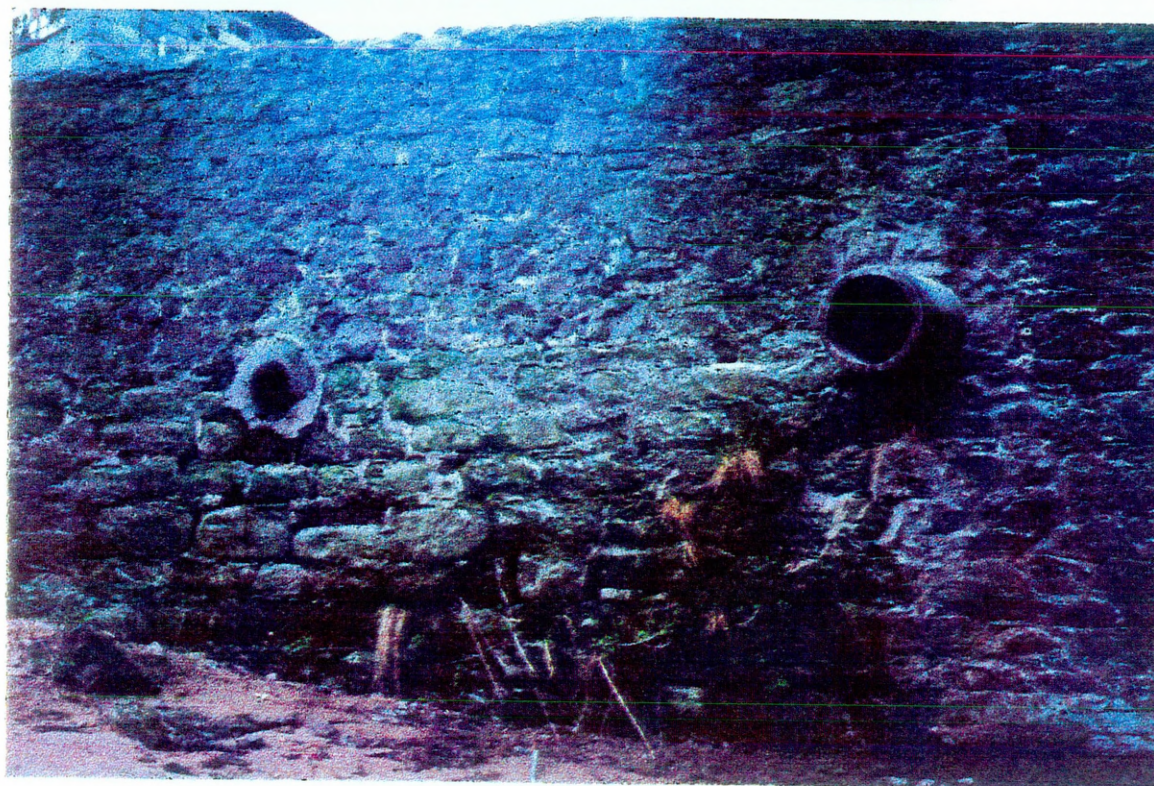


Plate 2 - Overflow Pipes (Bridgend)



Plate 3 Single Sided Weir Overflow (Bridgend)

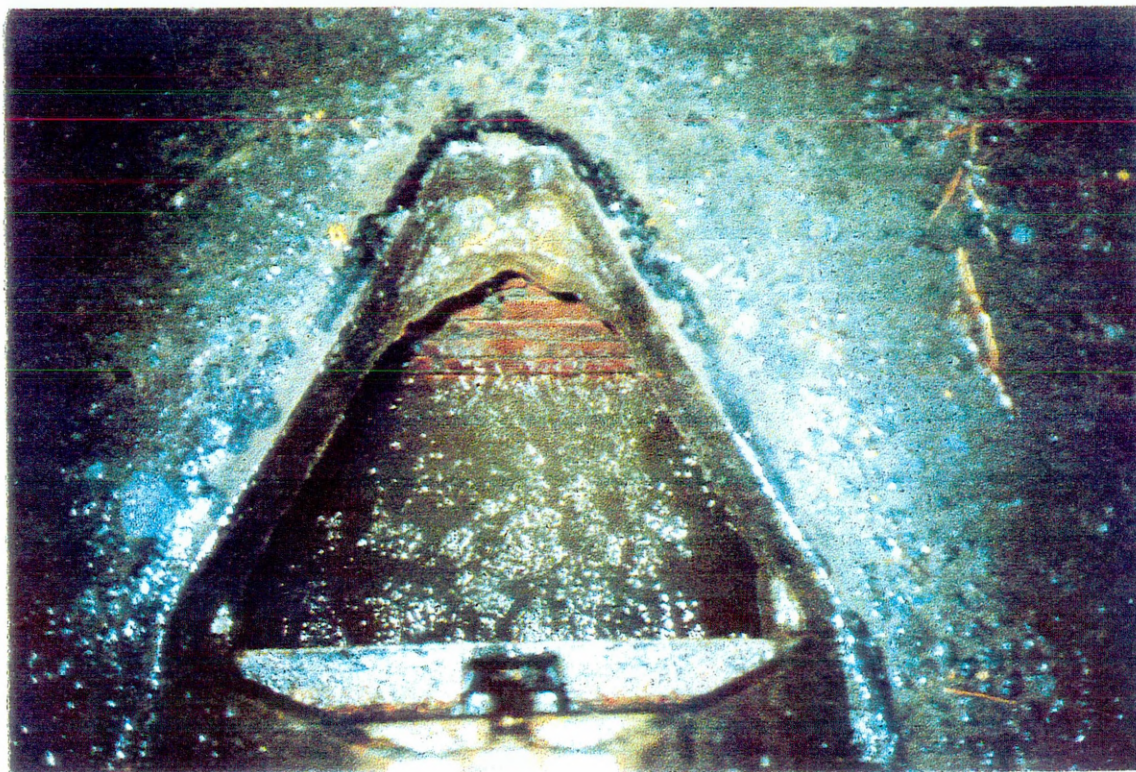


Plate 4 High Flows in Perth Sewerage System



Plate 5 - Low Volumetric River Flows



Plate 6 - Aesthetic Pollution

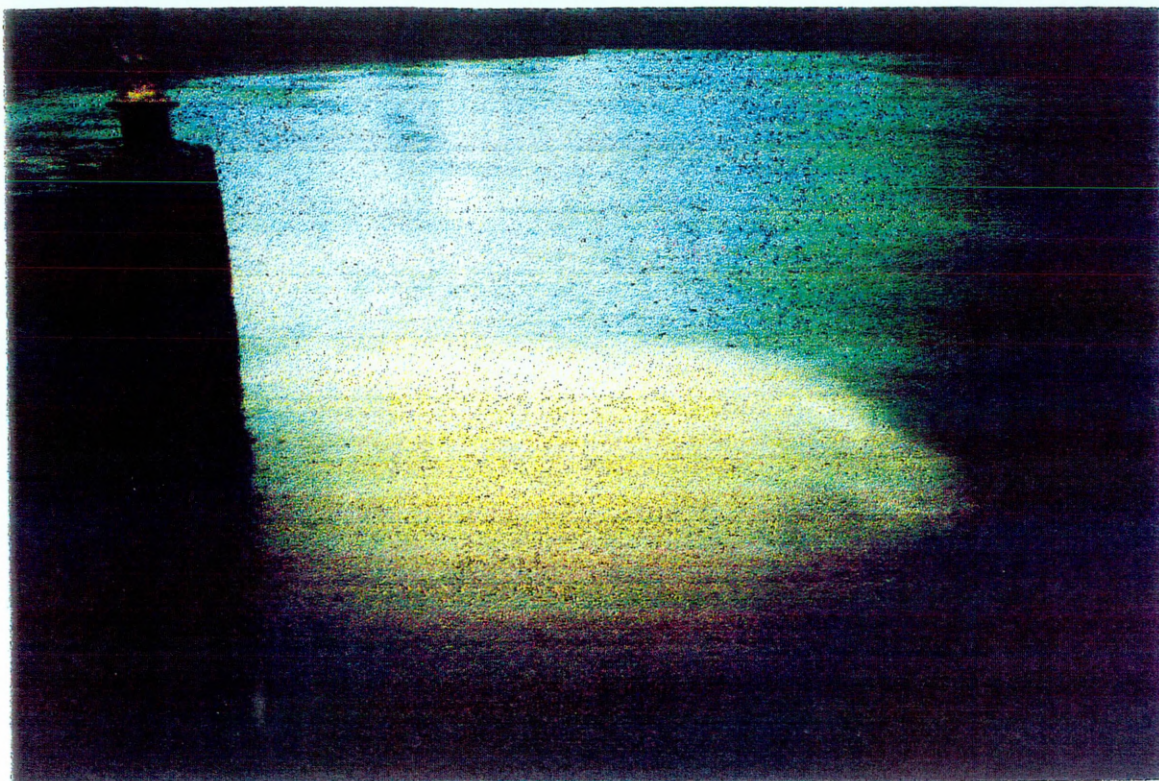


Plate 7 - South Inch Overflow Operation



Plate 8 - Sludge Injection - Friarton Pumping Station

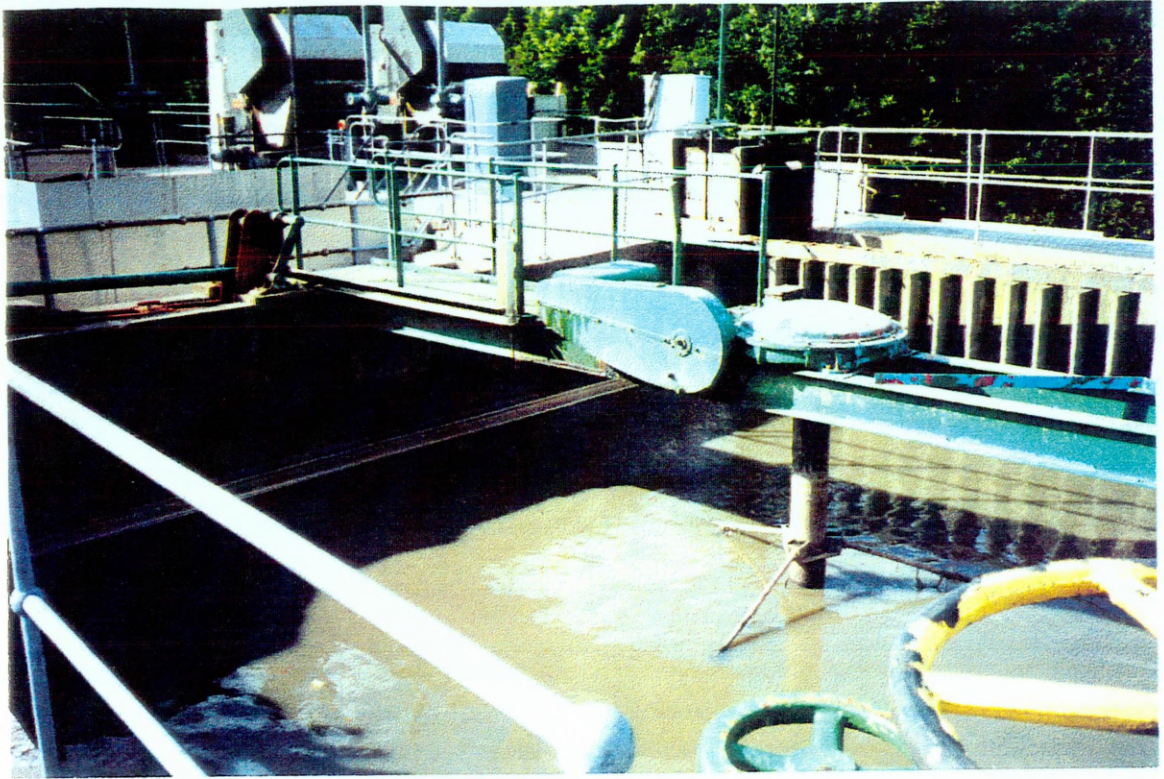


Plate 9 - Pre-Treatment at Sleepless Inch WTP



Plate 10 - Primary Settlement at Sleepless Inch WTP



Plate 11 - 3 Lanes of Settled Sewage

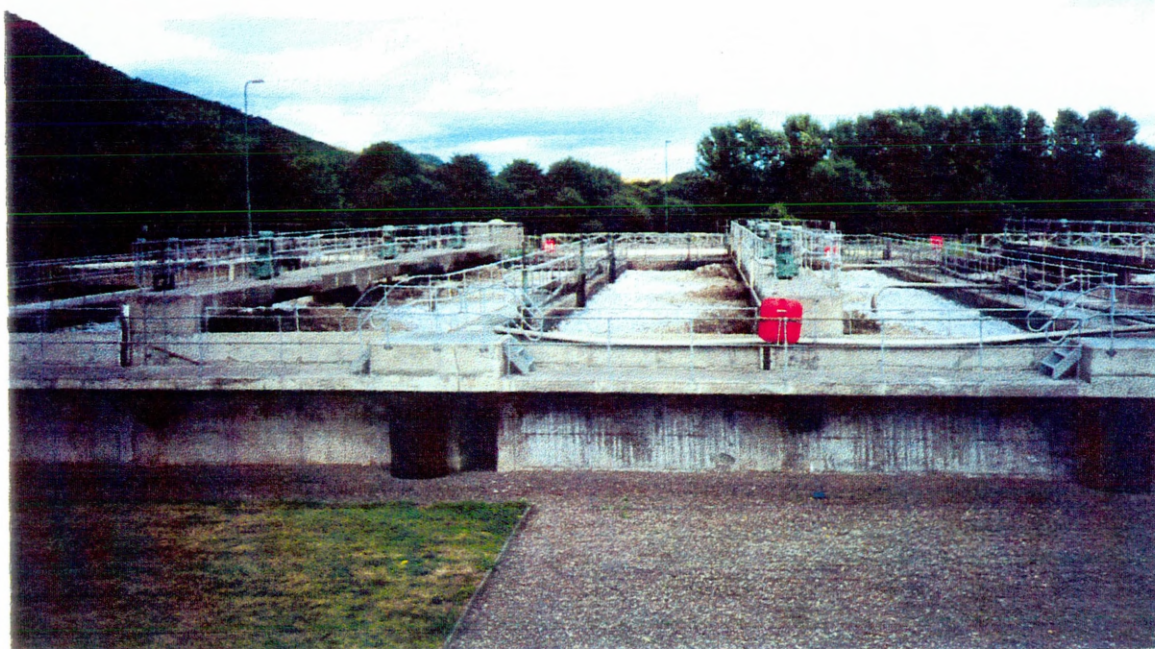


Plate 12 - Activated Sludge Plant

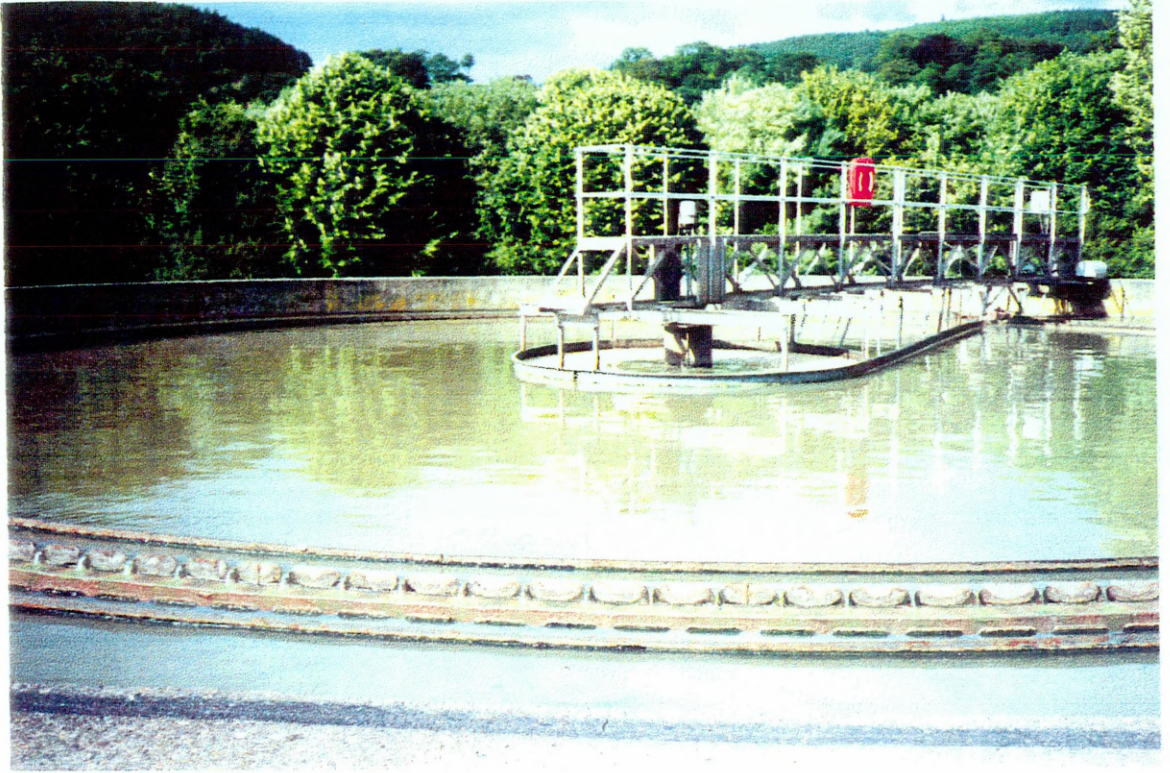


Plate 13 - Secondary Settlement Unit - Sleepless Inch

Appendix B

Hydroworks - QM Quality Model Construction and Verification - Storms

Contents

B.1	Introduction	B5
B.2	Deposited Pipe Sediment	B5
B.3	Quality Model Verification	B7
B.4	Summary	B30

List of Figures

B.1a	Burghmuir Rainfall Profile (24/8/95)	B7
B.1b	Perth Grammar Rainfall Profile (24/8/95)	B7
B.1c	Murray Royal Rainfall Profile (24/8/95)	B7
B.1d	Modelled Flow - Bridgend 24/8/95	B8
B.1e	Modelled Velocity - Bridgend 24/8/95	B8
B.1f	Modelled and Observed TSS - Bridgend 24/8/95	B9
B.1g	Modelled and Observed COD - Bridgend 24/8/95	B9
B.1h	Modelled and Observed Ammonia - Bridgend 24/8/95	B9
B.1i	Modelled Flow (South Inch Outfall – 24/8/95)	B10
B.1j	Modelled Velocity (South Inch Outfall – 24/8/95)	B10
B.1k	Modelled and Observed TSS - South Inch Outfall 24/8/95	B11
B.1l	Modelled and Observed COD - South Inch Outfall 24/8/95	B12
B.1m	Modelled and Observed Ammonia - South Inch Outfall 24/8/95	B12
B.2a	Burghmuir Rainfall Profile (29/8/95)	B12
B.2b	Perth Grammar Rainfall Profile (29/8/95)	B13
B.2c	Murray Royal Rainfall Profile (29/8/95)	B13
B.2d	Modelled Flow (South Inch 24/8/95)	B13
B.2e	Modelled Velocity (South Inch 24/8/95)	B13

List of Figures

B.2f	Modelled and Observed TSS - South Inch Outfall 29/8/95	B14
B.2g	Modelled and Observed COD – South Inch Outfall 29/8/95	B14
B.2h	Modelled and Observed Ammonia - South Inch Outfall 29/8/95	B15
B.3a	Burghmuir Rainfall Profile (31/5/95)	B15
B.3b	Perth Grammar Rainfall Profile (31/5/95)	B15
B.3c	Murray Royal Rainfall Profile (31/5/95)	B16
B.3d	Modelled Velocity (Bridgend – 31/5/95)	B16
B.3e	Modelled Flow (Bridgend – 31/5/95)	B16
B.3f	Modelled and Observed TSS - Bridgend 31/5/95	B18
B.3g	Modelled and Observed COD - Bridgend 31/5/95	B18
B.3h	Modelled and Observed Ammonia - Bridgend 24/8/95	B18
B.4a	Modelled Flow (Craigie – 31/5/95)	B19
B.4b	Modelled Velocity (Craigie – 31/5/95)	B19
B.4c	Modelled and Observed TSS - Craigie 31/5/95	B20
B.4d	Modelled and Observed COD - Craigie 31/5/95	B20
B.4e	Modelled and Observed Ammonia - Craigie 31/5/95	B21
B.4f	Modelled and Observed TSS – South Inch Outfall 31/5/95	B22
B.4g	Modelled and Observed COD – South Inch Outfall 31/5/95	B22
B.4h	Modelled and Observed Ammonia – South Inch Outfall 31/5/95	B23
B.5a	Burghmuir Rainfall Profile (17/10/95)	B23
B.5b	Perth Grammar Rainfall Profile (17/10/95)	B23
B.5c	Royal Murray Rainfall Profile (17/10/95)	B24
B.5d	Modelled Flow (North Inch – 17/10/95)	B24
B.5e	Modelled Velocity (North Inch – 17/10/95)	B24

List of Figures

B.5f	Modelled and Observed TSS – North Inch 17/10/95	B25
B.5g	Modelled and Observed COD – North Inch 17/10/95	B25
B.5h	Modelled and Observed Ammonia – North Inch 17/10/95	B26
B.6a	Modelled Flow (Craigie – 17/10/95)	B26
B.6b	Modelled Velocity (Craigie – 17/10/95)	B26
B.6c	Modelled and Observed TSS – Craigie 17/10/95	B27
B.6d	Modelled and Observed COD – Craigie 17/10/95	B27
B.6e	Modelled and Observed Ammonia– Craigie 17/10/95	B28
B.7a	Modelled Flow (South Inch Outfall – 17/10/95)	B28
B.7b	Modelled Velocity (South Inch Outfall – 17/10/95)	B28
B.7c	Modelled and Observed TSS – South Inch Outfall 17/10/95	B29
B.7d	Modelled and Observed COD– South Inch Outfall 17/10/95	B30
B.7e	Modelled and Observed Ammonia – South Inch Outfall 17/10/95	B30

Tables

B.1	Storm Data Collection Appraisal	B5
B.2	Accuracy of Modelled Predictions 24/8/95	B19
B.3	Accuracy of Modelled Predictions 31/5/95	B19

Hydroworks -QM

Construction and Verification - Storm

B.1 Introduction

Chapter ten described the calibration of the Perth Hydroworks -QM model for dry weather conditions. This appendix details the construction of the model under storm conditions. The additional work required for storm modelling was the designation of surface washoff pollutant indices for each land-use type. Default data were utilised for this.

B.2 Deposited Pipe Sediment

Hydroworks -QM gave no consideration to deposited sediment pollutant characteristics. Consequently, no sediment conversion work was necessary to upgrade from MOSQUITO to -QM. Sediment depths however still required to be defined within pipe lengths where sediment was present. These sediment deposits were inerodible and served only to limit the hydraulic capacity of the pipes. Storm calibrations plots using the -QM package are shown in figs. B.1 – B.7.

Table B.1 - Storm Data Collection Appraisal:-

Date	Location & Number of Samples	Rain Gauge – Murray Royal (Peak & Vol.)	Rain Gauge - Burghmuir (Peak & Vol.)	Rain Gauge - Perth Grammar (Peak & Vol.)
17/5/95	Bridgend - 24 South Inch - 13 Moncrieffe - 24 Craigie - 24	3 mm/hr 1 mm	14 mm/hr 5.6 mm	18 mm/hr 4.2 mm
31/5/95	Bridgend - 24 South Inch - 13 Moncrieffe - 24 Craigie - 24	24 mm/hr 7.4 mm	45 mm/hr 5.8 mm	7 mm/hr 2.2 mm
20/7/95	Craigie - 24	18 mm/hr 4.4 mm	18mm/hr 5.2 mm	13 mm/hr 5 mm
24/8/95	Bridgend - 22 South Inch - 23	1.5 mm/hr 1.6 mm.	1.6 mm/hr 0.8 mm	1.7 mm/hr 1.2 mm

Table B.1 (Cont.)

Date	Location & Number of Samples	Rain Gauge – Murray Royal (Peak & Vol.)	Rain Gauge - Burghmuir (Peak & Vol.)	Rain Gauge - Perth Grammar (Peak & Vol.)
29/8/95	South Inch - 24	1.2 mm/hr 1.2 mm	1.6 mm/hr 0.8 mm	1.7 mm/hr 0.4 mm
17/10/95	North Inch - 24 South Inch - 24 Craigie - 24	12 mm/hr 3.2 mm	6.4 mm/hr 0.4 mm	6.5 mm/hr 1.4 mm

The 17/5 and 31/5 events show significant spatial variation, as shown in table B.1. The former event required to be discarded for quality modelling purposes as the underlying hydraulic model could not be verified. This is believed to be a direct consequence of the spatial variation. Greater modelling accuracy was achieved for the 31/5 event although sampling problems occurred at the South Inch resulting in the loss of important quality data.

A significant rainfall event occurred on the 20/7, however no personnel were on site during the rainfall event. The sampling site in Craigie, which was automatically operated provided a complete set of twenty four samples. It subsequently became apparent however, that a weaker intensity event in the early hours of the morning may have caused the sampler to begin its program cycle. Consequently the samples obtained might not correspond to the large intensity event. This reinforced the necessity of being on site prior to any rainfall event.

The 24/8 and 29/8 events are of very low intensity, and do not reach above 2mm/hr at any of the logging sites. Nevertheless, a substantial amount of useful qualitative data was gathered for these two events.

The 17/10 event was of greater magnitude than the 24/8 & 29/8 events. Data were collected from three sites; North Inch, South Inch and Craigie. Hydraulic verification problems were encountered however, at the South Inch Pumping Station as logged flow proved somewhat larger than the model was suggesting. The problem was attributed to a logger calibration problem, as prior to the over predictions a half hour period passed where no data were recorded. This is believed to be a result of ragging. When logger function re-occurred the data obtained were consistently high.

B.3 Quality Model Verification

The results shown were obtained from the model which took no account of infiltration. Storm quality modelling using the -QM “infiltration” model formed part of the sensitivity testing and is discussed in chapter twelve.

The rainfall profiles for the 24/8/95 event are shown in figures B.1a, B.1b and B.1c. Despite this event being of low intensity it was still used for verification purposes as the resulting flows were not dissimilar to DWF. Consequently, the event was used to analyse the integrity of the model when flows only marginally drifted from dry weather conditions.

Fig. B.1a Burghmuir Rainfall Profile (24/8/95)

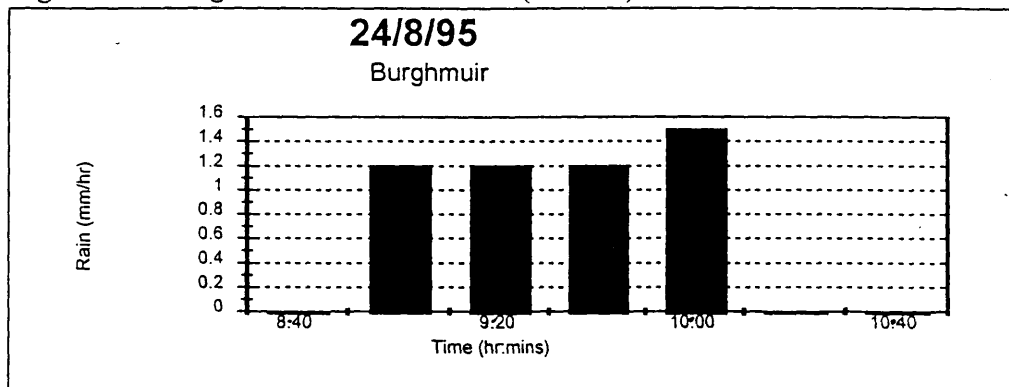


Fig. B.1b Perth Grammar Rainfall Profile (24/8/95)

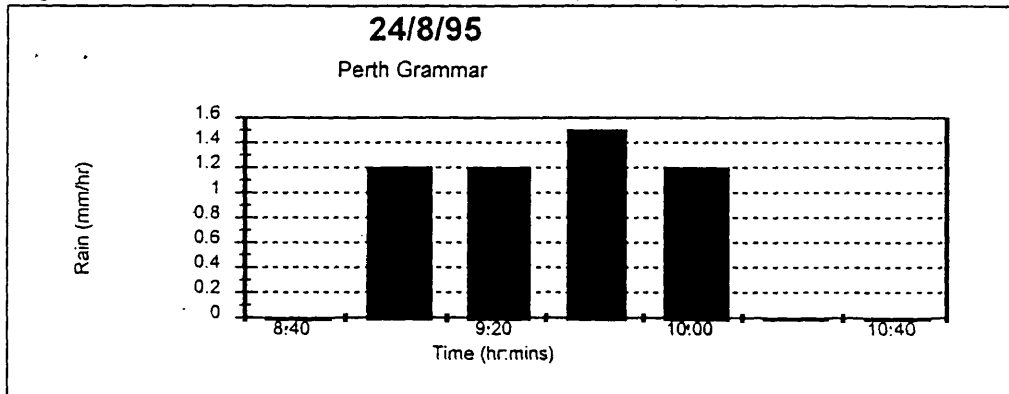


Fig. B.1c Murray Royal Rainfall Profile (24/8/95)

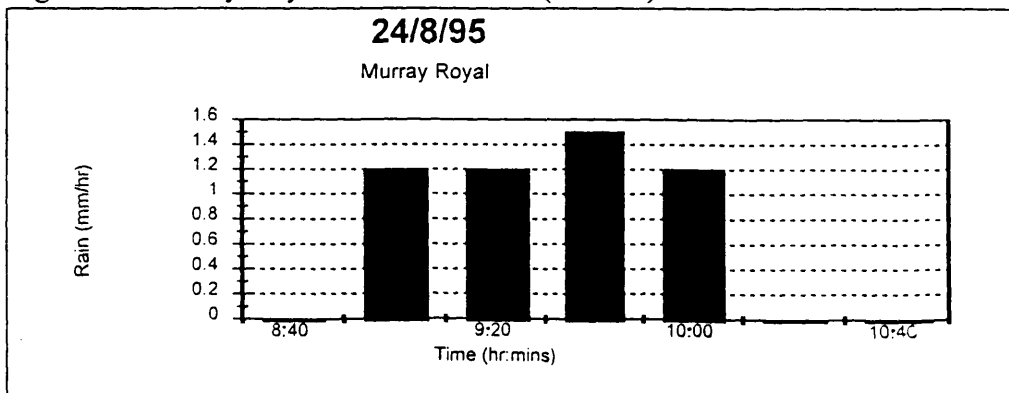


Fig. B.1d Modelled Flow - Bridgend 24/8/95

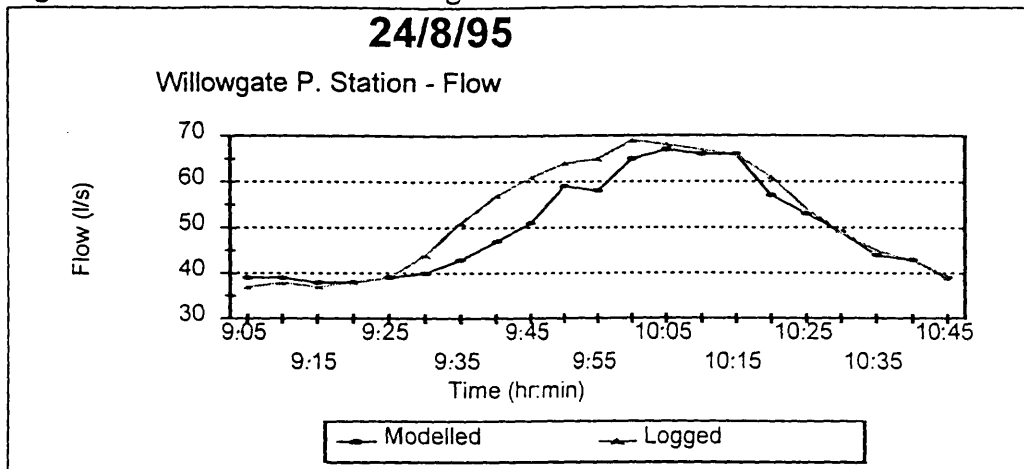
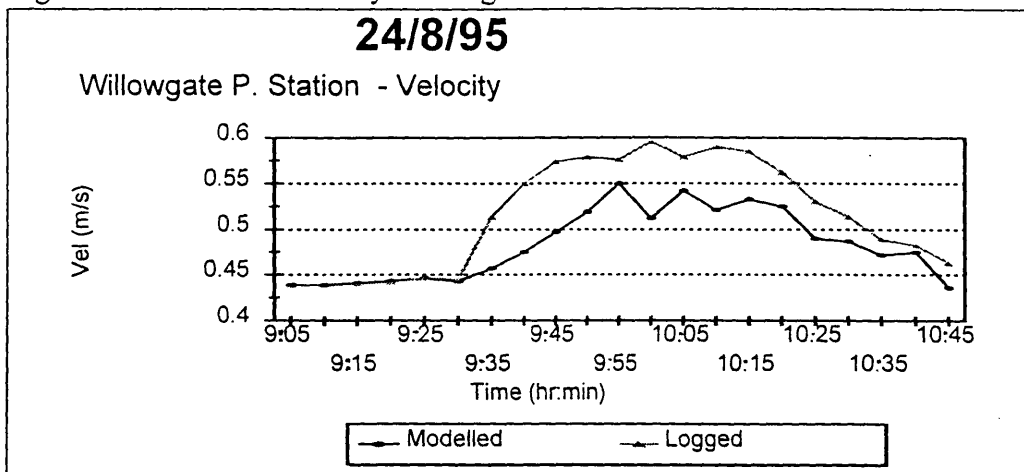


Fig. B.1e Modelled Velocity - Bridgend 24/8/95



Figures B.1d and B.1e show the hydraulic modelling in the Bridgend subcatchment. Volume was under predicted by 4.5% and peak flow was under predicted by 3%. Velocity was under predicted by 15% when the peak flow occurred. Figures B.1f-B.1h show the corresponding qualitative plots.

Quality Modelling - Bridgend (24/8):

Fig. B.1f shows poor modelling of TSS although COD is modelled substantially better (figure B.1g). The modelled error with respect to the total TSS load was calculated as -47%, whereas, the modelled error with respect to the total COD load was only -8%. QM did not predict a flush for either determinand. The modelled error at the time of the observed peak flush (approx. 9.55am) was -65% for TSS and -23% for COD. Problems occurred with the modelling of ammonia (fig. B.1h) and calibrations were attempted. Unfortunately, no acceptable profile could be obtained without a force fitting of the model. The modelled error with respect to the total ammonia load was calculated as +65%. No analysis was

attempted to determine the peak error for ammonia as this determinand undergoes dilution during storm conditions.

Fig. B.1f Modelled and Observed TSS - Bridgend 24/8/95

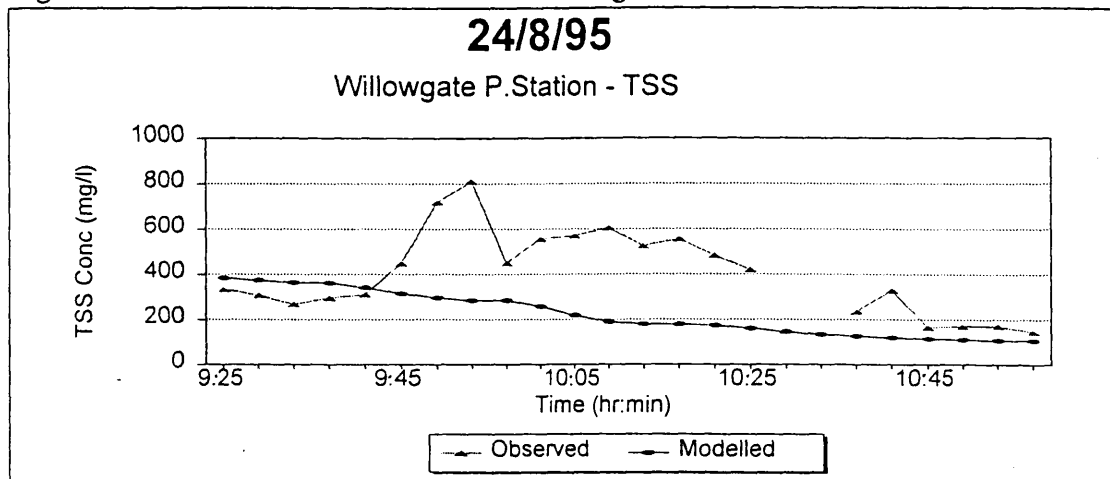


Fig. B.1g Modelled and Observed COD - Bridgend 24/8/95

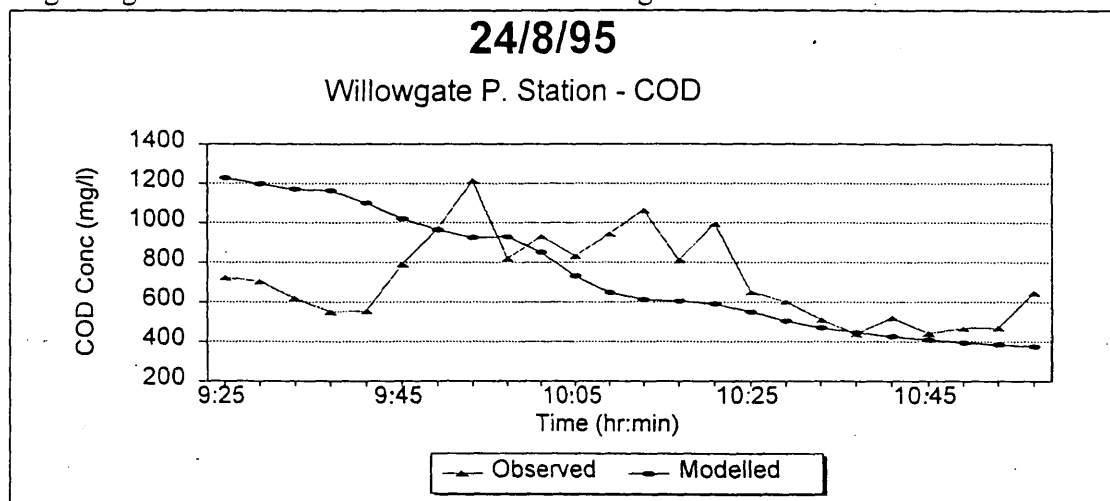


Fig. B.1h Modelled and Observed Ammonia - Bridgend 24/8/95

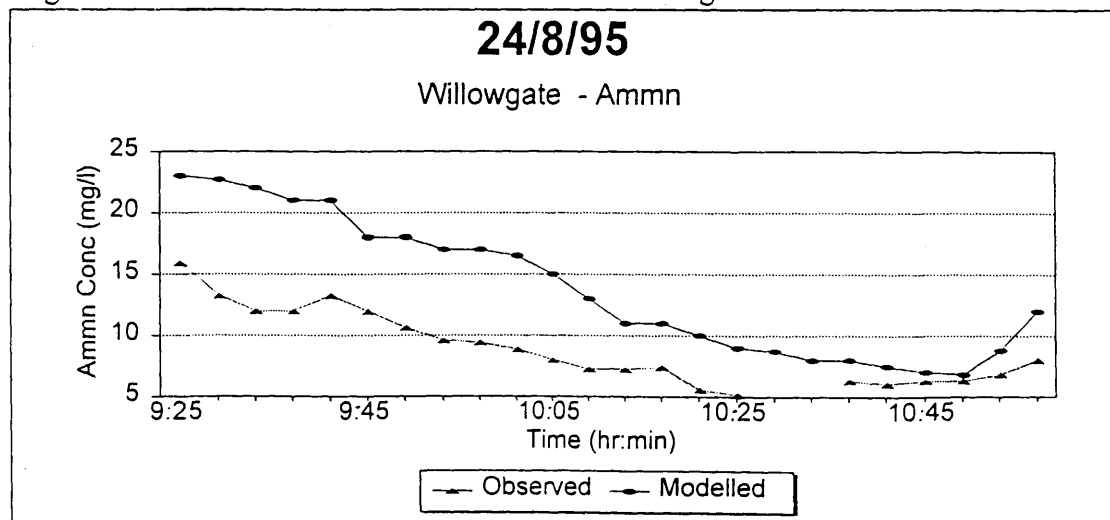


Fig. B.1i Modelled Flow (South Inch Outfall – 24/8/95)

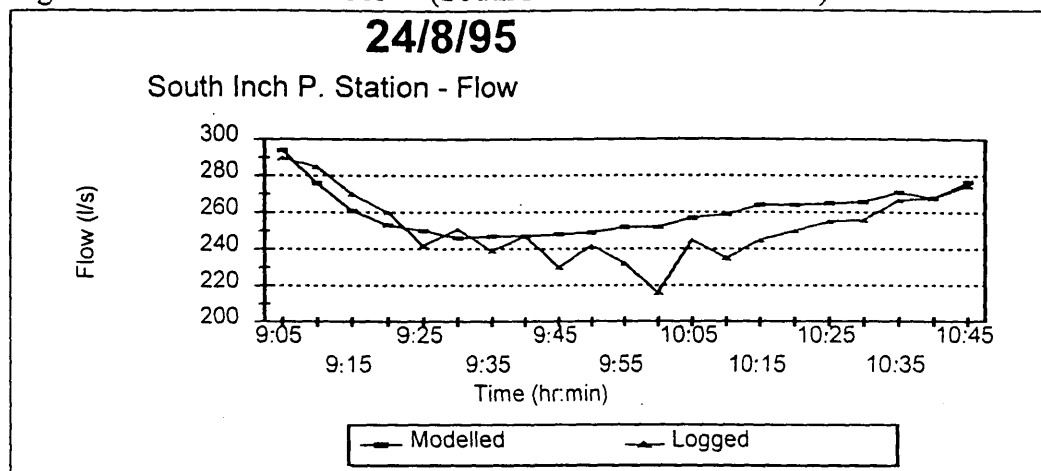
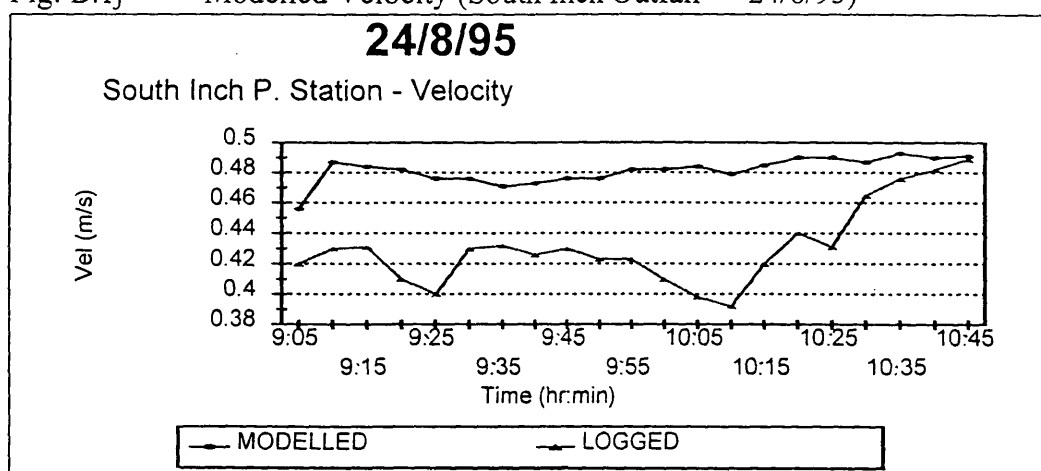


Fig. B.1j Modelled Velocity (South Inch Outfall – 24/8/95)



Hydraulic Modelling - South Inch (24/8):

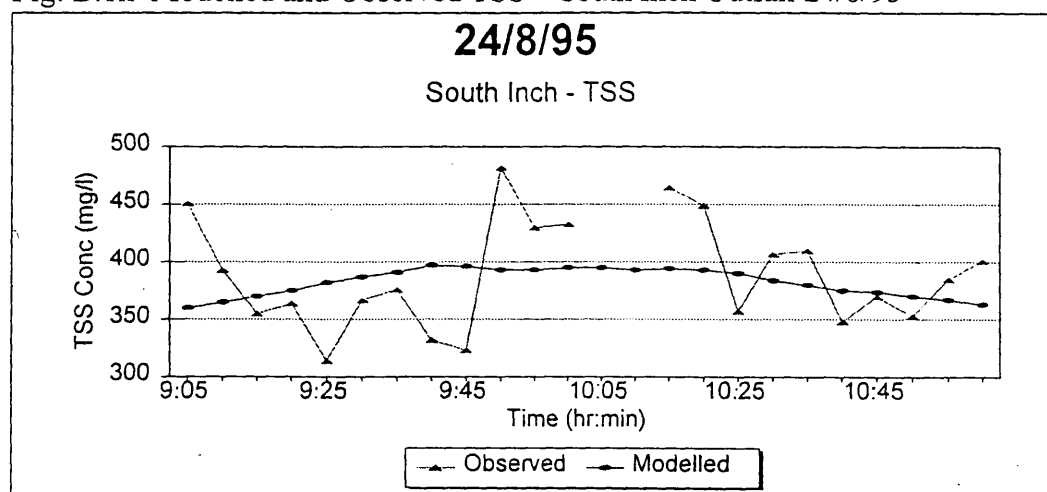
Figures B.1i & B.1j show the modelling of flow and velocity profiles at the South Inch outfall. Volume was over predicted by 3% and the average modelled error with respect to velocity was +12.5%. No analysis was carried out with respect to the "peak error" as the flows decreased over the survey period.

Quality Modelling - South Inch (24/8)

Figure B.1k compares the modelled and observed TSS profiles for the 24/8 event at the South Inch sampling site. The modelled error with respect to total TSS load was calculated as +11%. The maximum error with respect to concentration occurred at approximately 9.05am. This error was calculated as +25%. The peak observed concentration is seen to be 398mg/l at 9.40am. It is debatable whether the model suffered from a time lag, however the modelled peak occurred 10 minutes later and was 482mg/l. Based on these figures the error with respect to the peak was +21%. With reference to figure B.1k, it can be seen that although the modelled suspended solids agree in magnitude with the observed data, the

actual trend has not been represented. The most probable reason for this is because of the problems and uncertainties associated with quality data collection (Uhl, 1993)¹. As flow rate does not fluctuate dramatically, it would therefore be expected that the TSS profile would remain relatively constant. However sampled data was erratic and had no real trend. Consequently, given the potential magnitude of sampling (and modelling) error it was reasonable to assume that a representation of reality had, in fact, been made. The modelled error with respect to the total COD load was +12%. The maximum error with respect to concentration was +36%. The peak sampled concentration was 1194mg/l at 10:15am, whereas the modelled peak was 862mg/l (fig. B.11). No time lag was evident. This produced an error of -28%. The total ammonia load was over predicted by +23%. The modelled and observed ammonia profiles can be seen in figure B.1m.

Fig. B.1k Modelled and Observed TSS - South Inch Outfall 24/8/95



1. Uhl, M. (1993) Uncertainty of Quantity and Quality Data Measured in a Combined Sewer System. Sixth International Conference on Urban Storm Drainage.

Fig. B.11 Modelled and Observed COD - South Inch Outfall 24/8/95

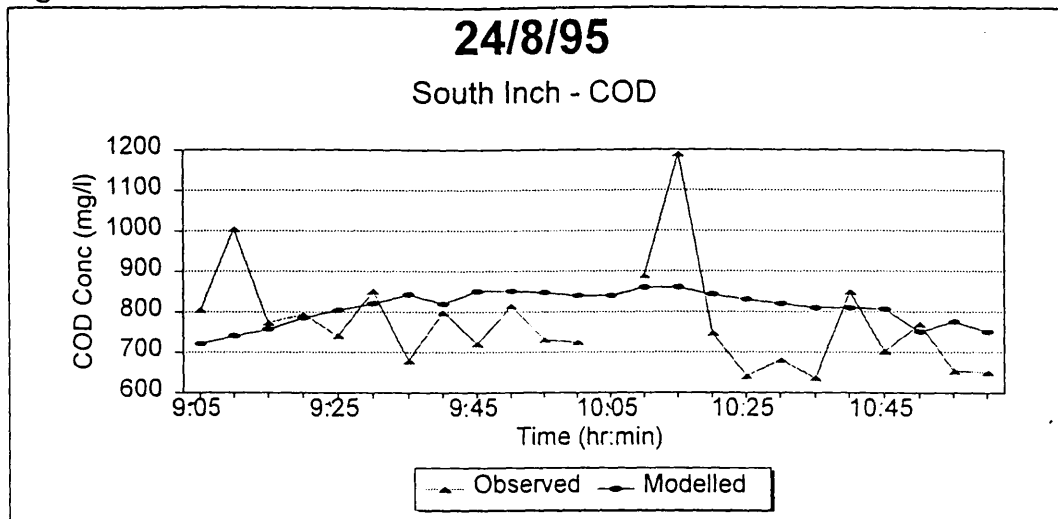


Fig. B.1m Modelled and Observed Ammonia - South Inch Outfall 24/8/95

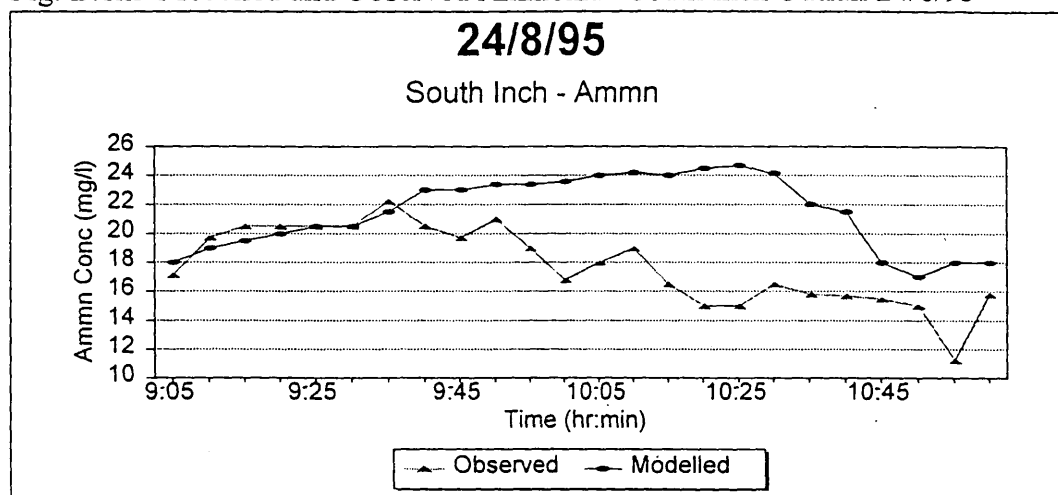


Fig. B.2a Burghmuir Rainfall Profile (29/8/95)

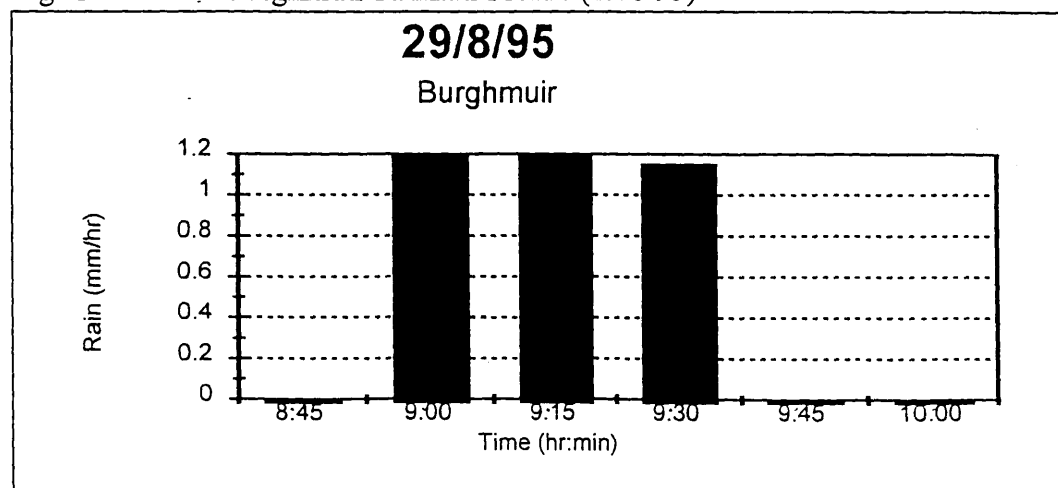


Fig. B.2b Perth Grammar Rainfall Profile (29/8/95)

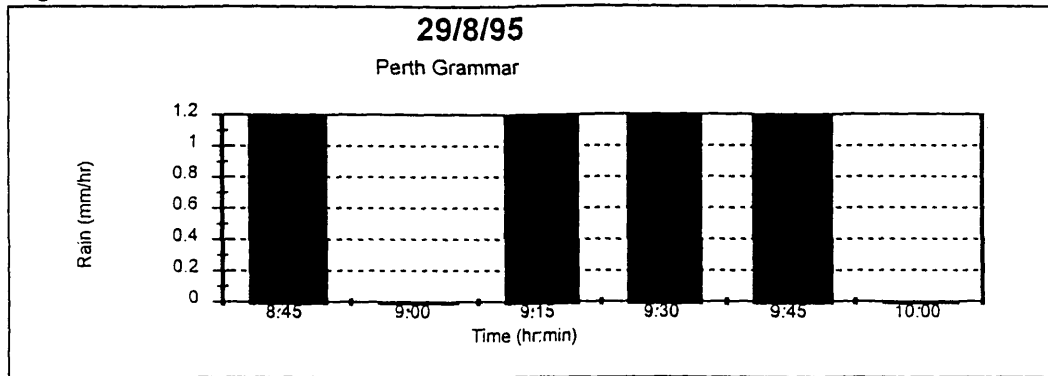


Fig. B.2c Murray Royal Rainfall Profile (29/8/95)

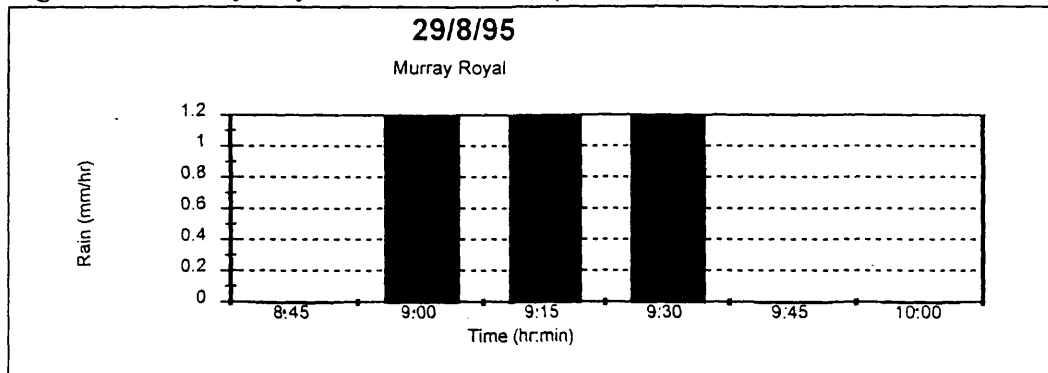


Fig. B.2d Modelled Flow (South Inch 24/8/95)

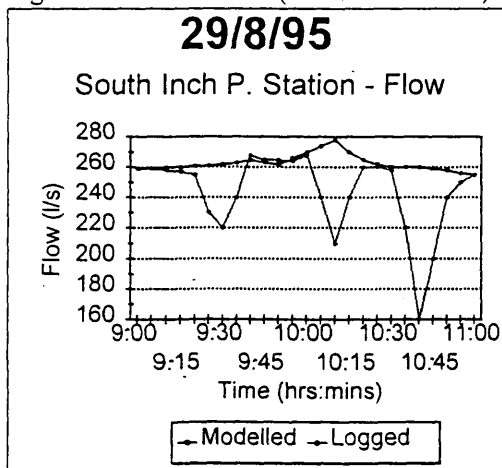
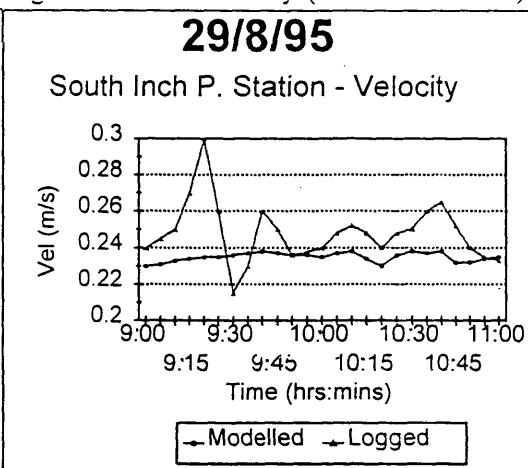


Fig. B.2e Modelled Velocity (South Inch 24/8/95)



Figures B.2d and B.2e show the modelling of flow and velocity at the South Inch outfall for the 29/8 event. Volume was over-predicted by +7% and peak flow was over-predicted by 33%. Velocity was under predicted by 4.7% when the peak flow occurred. The average modelled error with respect to velocity was -5%. As can be seen from figs. B.2d & B.2e certain peaks and troughs are not matched. It is possible that this anomaly is due to refined pump operation (on off control) which has not been represented to the degree of accuracy required.

Figure B.2f shows that -QM in general, has over predicted TSS. The modelled total TSS load error was calculated as +31%. A peak sampled concentration of

320mg/l occurred at 9.40am. This peak was not picked up by the model. As it was not believed a time lag was present, the modelled error with respect to the peak was taken as -18%. This figure was derived using the corresponding modelled data at 9.40am. It is apparent that for this particular event, -QM would benefit from calibration. However, by decreasing suspended solids, COD would also be reduced ($COD_{total} = COD_{dissolved} + COD_{sediment-attached}$) and a reduction in COD concentration was not warranted either for this event (fig. B.2g) as the total COD load error was +5% (peak error of -53%), or for the event on the 24/8/95. As the rainfall events on the 24/8/95 and 29/8/95 are of such low intensity (figs. B.1 a,b,c & B.2 a,b,c), the resulting flows are similar to the peak dry weather flows. The accuracy of the qualitative storm plots are therefore similar to the accuracy obtained during dry weather quality modelling. During more extreme rainfall, however the modelled outputs drifted somewhat from reality.

Fig. B.2f Modelled and Observed TSS - South Inch Outfall 29/8/95

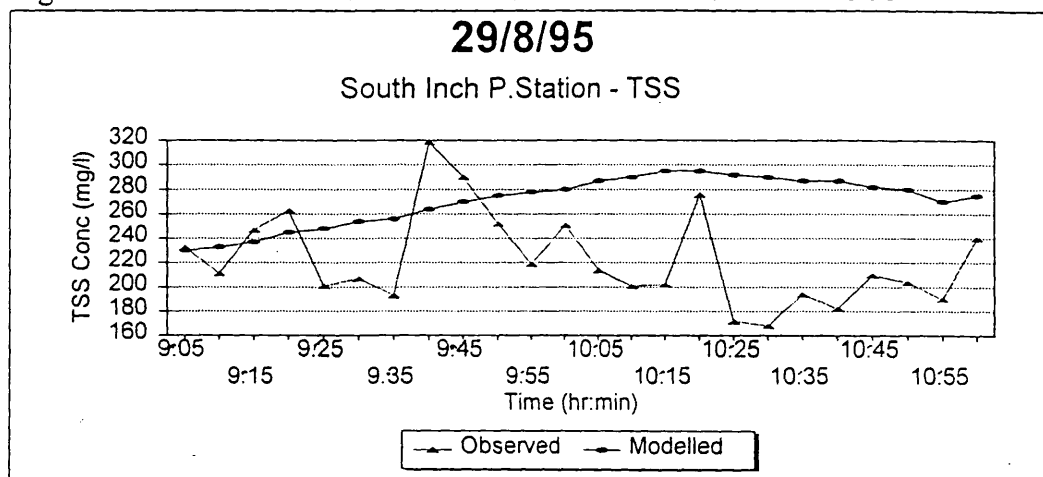


Fig. B.2g Modelled and Observed COD - South Inch Outfall 29/8/95

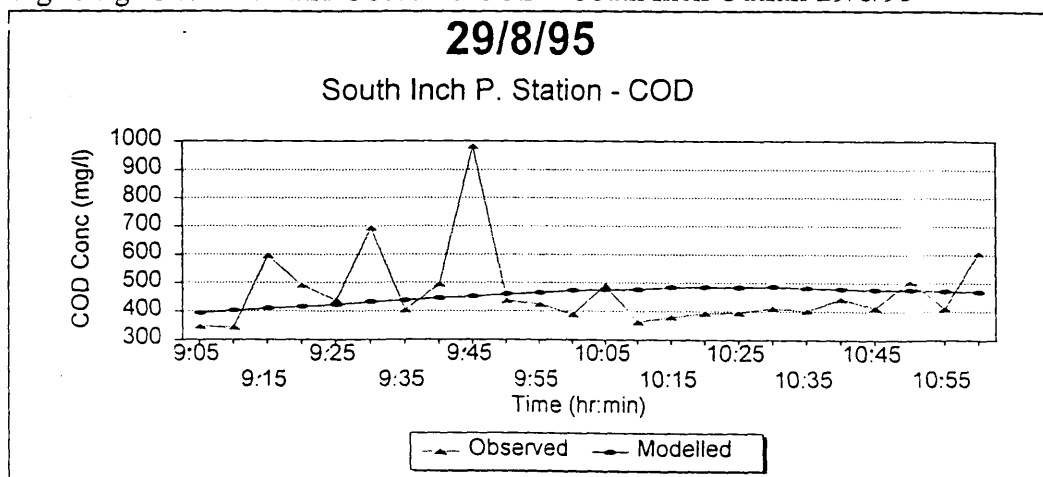


Fig. B.2h Modelled and Observed Ammonia - South Inch Outfall 29/8/95

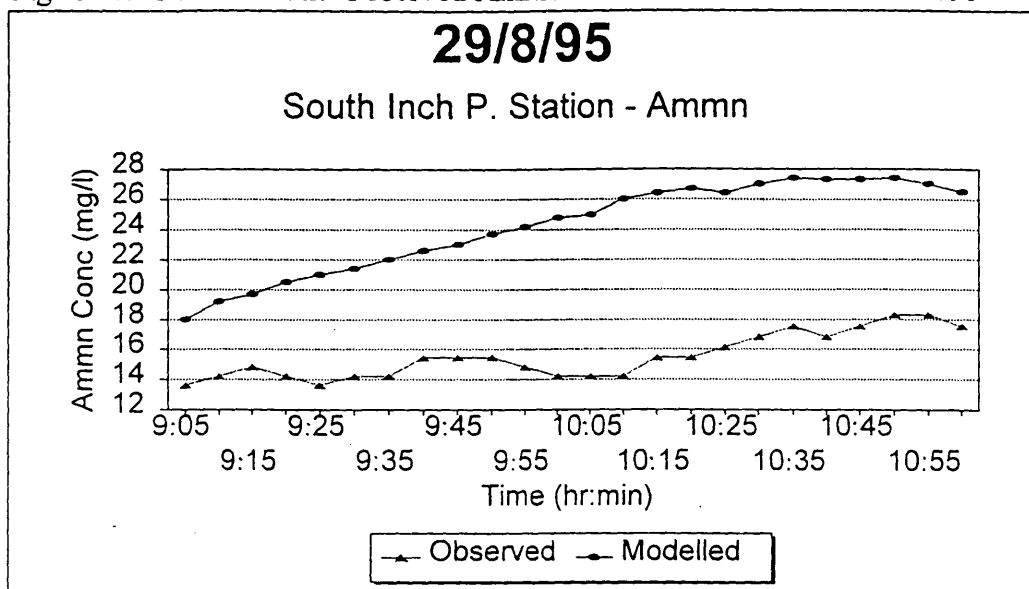


Fig. B.3a Burghmuir Rainfall Profile (31/5/95)

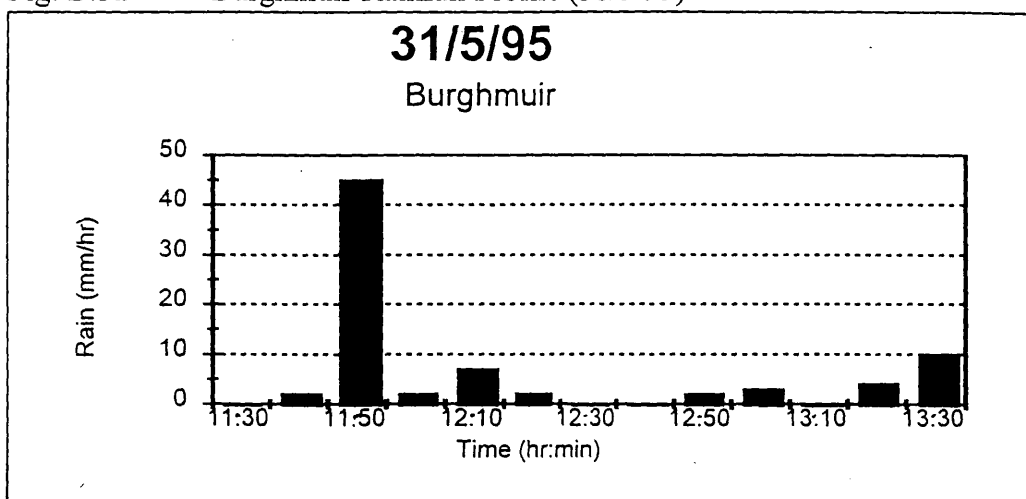


Fig. B.3b Perth Grammar Rainfall Profile (31/5/95)

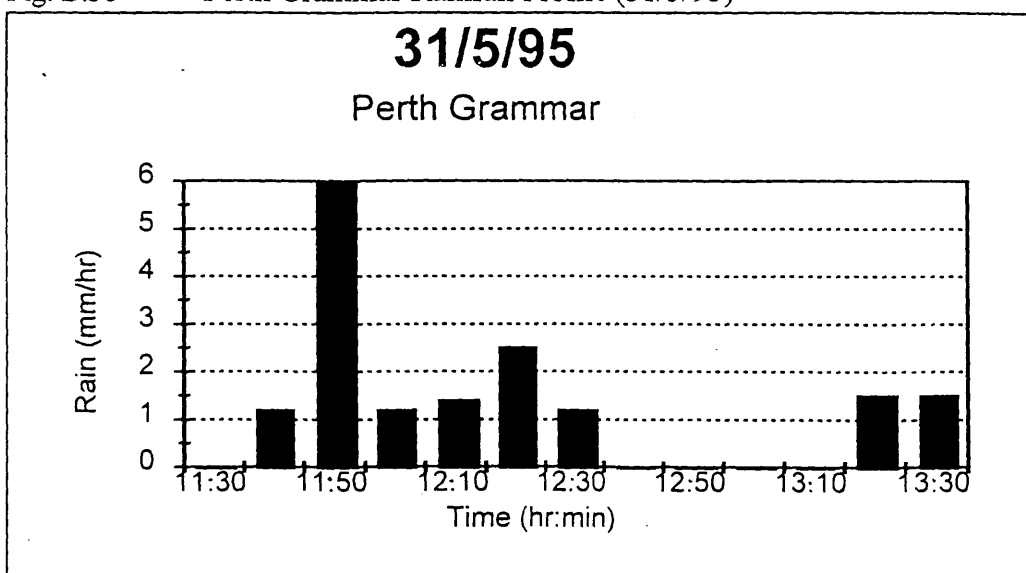


Fig. B.3c Murray Royal Rainfall Profile (31/5/95)

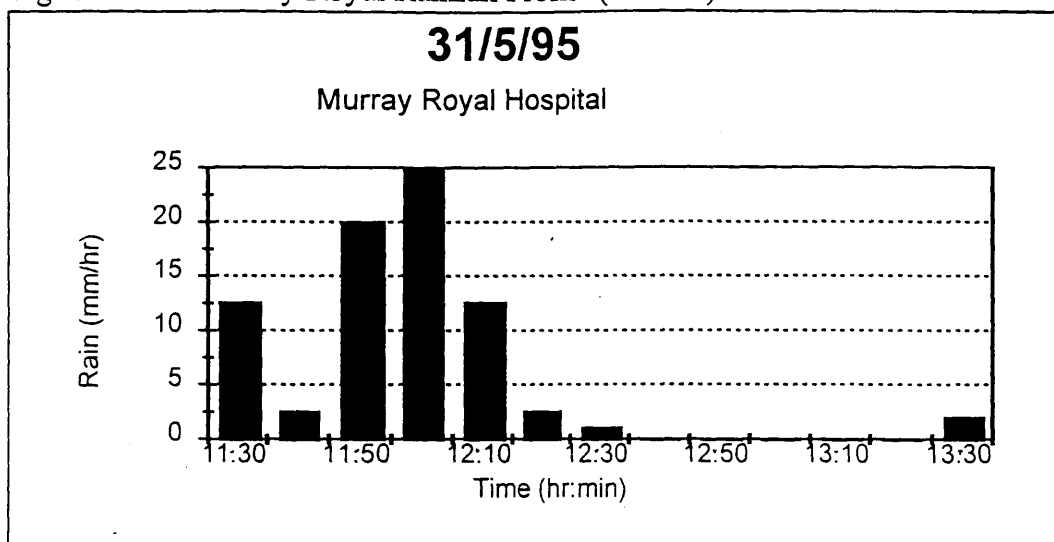


Fig. B.3d Modelled Velocity (Bridgend - 31/5/95)

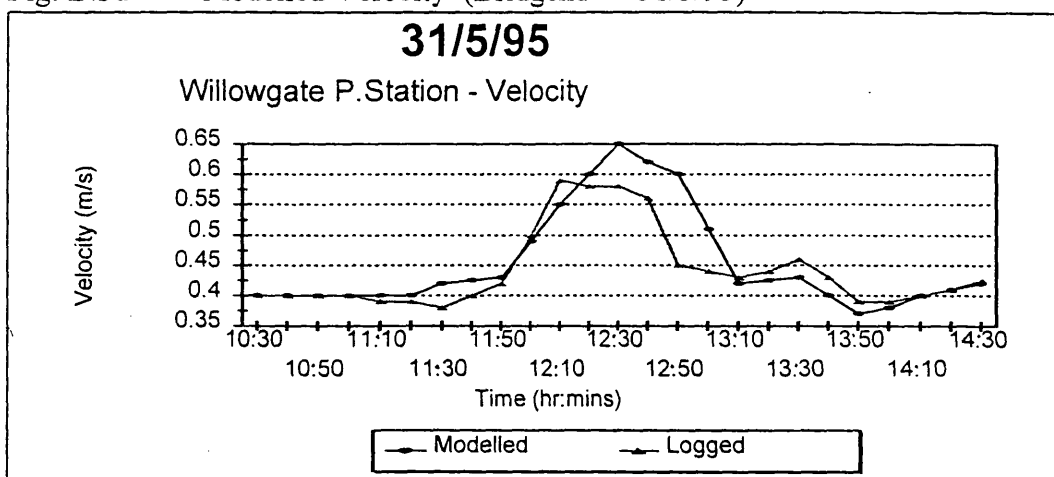
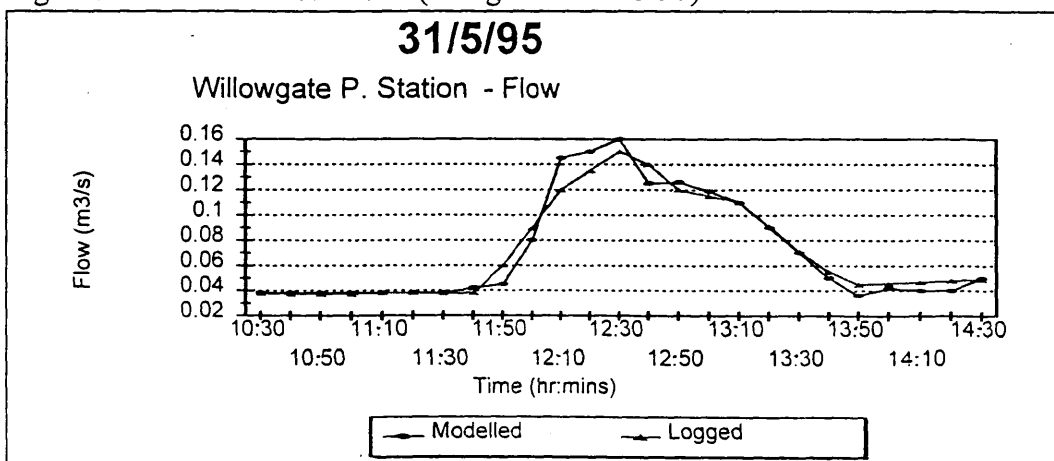


Fig. B.3e Modelled Flow (Bridgend - 31/5/95)



Comment:

As can be seen from the TSS plot (fig. B.3f), -QM has grossly underestimated this determinand. The modelled error with respect to total TSS load was calculated as - 82%. The modelled error with respect to the observed peak TSS

flush concentration (at approx. 11.52am) was calculated as -88%. Nevertheless, with reference to velocity and flow profiles it is apparent that the hydraulics of the Bridgend subcatchment are modelled accurately (figs B.3d & B.3e). Table 10.6 shows that volume is modelled to less than 3% of the observed data for this event. Consequently, it would be expected that a better qualitative profile would be obtained. Nevertheless, in this particular case flow rate has increased substantially (storm peak approximately 4x dwf) and thus it is hypothesised that as bed shear stresses have increased in this subcatchment from under 2N/m^2 (approx.) to 4N/m^2 (approx. calculations based on peak velocities), a portion of the deposited pipe sediment in the Bridgend sub-catchment may have been eroded (Wotherspoon, D, 1994)² As -QM gives no consideration to these processes, the model grossly underestimates the flush. This hypothesis is substantiated as the increase in sampled TSS concentration, which is not modelled correctly, occurs during the period of increasing flow rate. (It may be thought possible that the gross over prediction of TSS was a consequence of the sampler head being too low in the vertical suspended solid concentration gradient (Verbanck, M, 1993)³ i.e. the sampler has sampled sediment from the deposited sediment bed. This however could not have been the case for two main reasons. The first being that on-site observations during the data collection exercise showed that even slight increases in flow rate always resulted in the sampler head being carried towards the surface of the sewage column and the second reason being that deposited sediment is not present at the sampling site (it is only present upstream). Consequently, the poor modelling results were attributed to the limitations of the model. Tables B.2. and B.3 show the accuracy of the modelled predictions for events 24/8/95 and 31/5/95.

2. Wotherspoon, D.J.J.W. (1994) The Movement of Cohesive Sediment in a Large Combined Sewer. PhD Thesis. University of Abertay Dundee.

3. Verbanck, M. (1993) Identification of the Depth Dependent Transportation of Particulate Solids in Dry Weather Sewage Flows. Sixth International Conference on Urban Storm Drainage

Fig. B.3f Modelled and Observed TSS - Bridgend 31/5/95

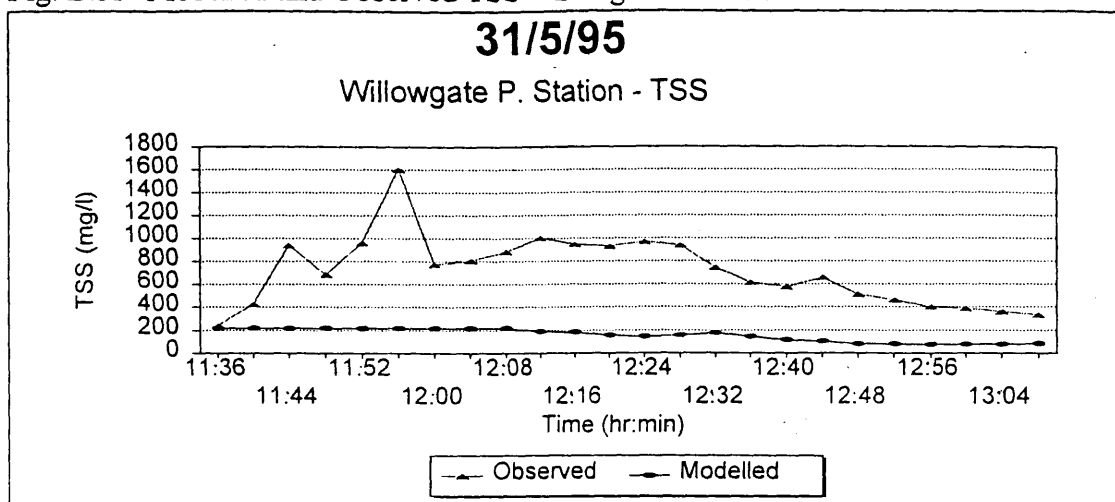


Fig. B.3g Modelled and Observed COD - Bridgend 31/5/95

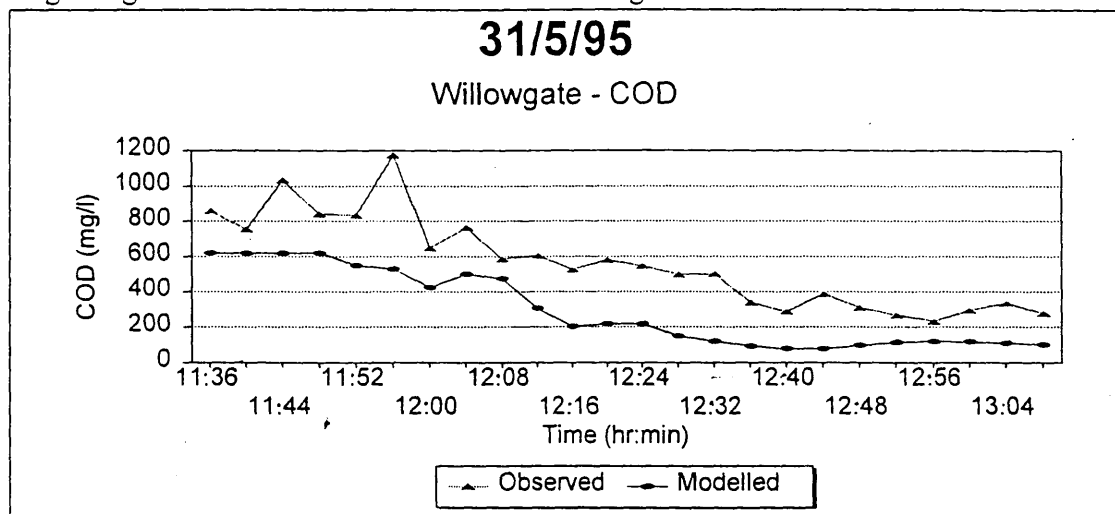


Fig. B.3h Modelled and Observed Ammonia - Bridgend 24/8/95

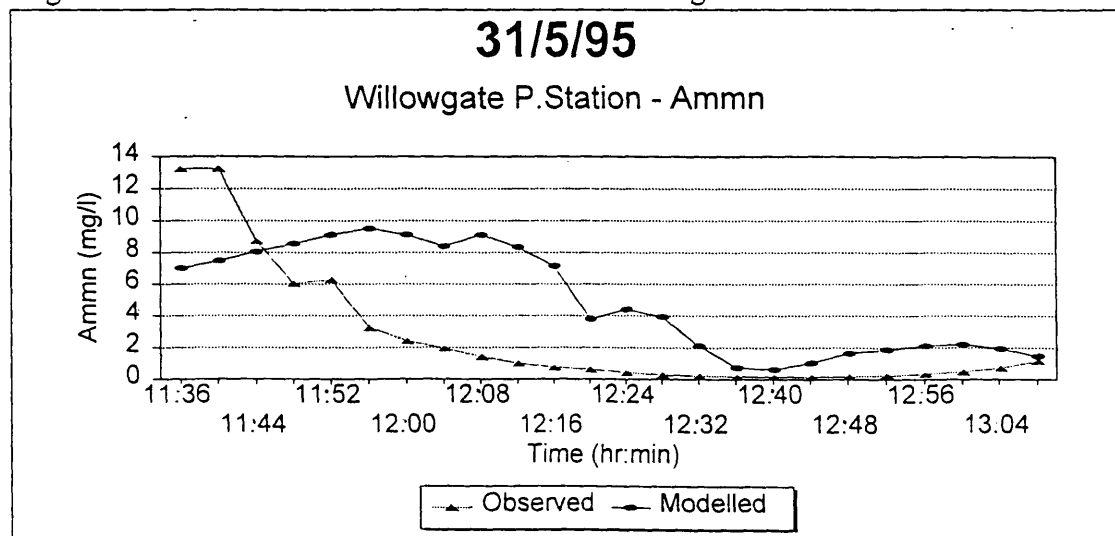


Table B.2 Accuracy of Modelled Predictions 24/8/95

Rainfall Event 24/8/95 Peak 1.5mm/hr Vol. 1.2mm Sampling Site: Willowgate	HYDROWORKS -QM	SAMPLED	ACCURACY of PREDICTION
Total TSS load (g/s)	237.3	452	-47%
Total COD load (g/s)	784.3	850.6	-8%

Table B.3 Accuracy of Modelled Predictions 31/5/95

Rainfall Event 31/5/95 Peak 25mm/hr Vol. 6.8mm Sampling Site: Willowgate	HYDROWORKS -QM	SAMPLED	ACCURACY of PREDICTION
Total TSS load (g/s)	346.5	1870.8	-82%
Total COD load (g/s)	674.7	1284.6	-48%

Fig. B.4a Modelled Flow (Craigie – 31/5/95)

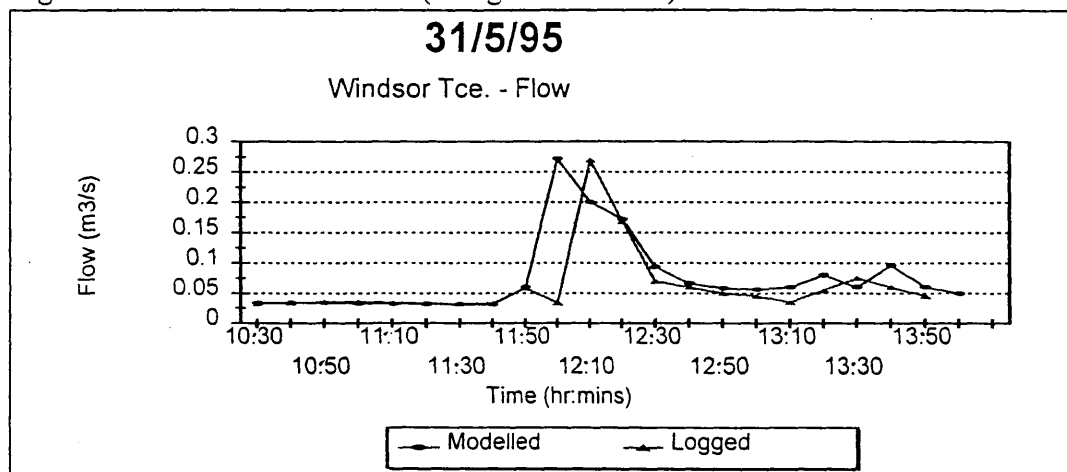
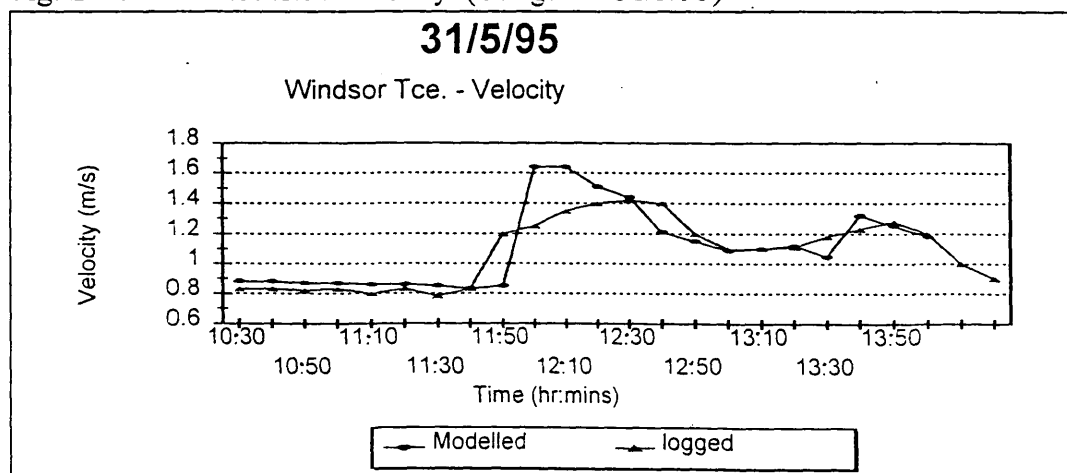


Fig. B.4b Modelled Velocity (Craigie – 31/5/95)



Comment:

Table 10.6 shows that the modelled error with respect to volume was +22% at the Craigie sampling site and that peak flows were over predicted by 3%. Velocity was over-predicted by 17% when the peak flow occurred. The average modelled error with respect to velocity was however calculated as only +2.4%. Figures B.4c and B.4d compare the modelled and observed TSS and COD profiles, respectively. The modelled error with respect to total TSS and COD load was -7% and +75%, respectively. The peak TSS concentration was under-predicted by 67% (12:36pm) whereas the peak COD concentration was under predicted by only 14% (12:24pm). The modelled error with respect to the total ammonia load was +191% (figure B.4e).

Fig. B.4c Modelled and Observed TSS - Craigie 31/5/95

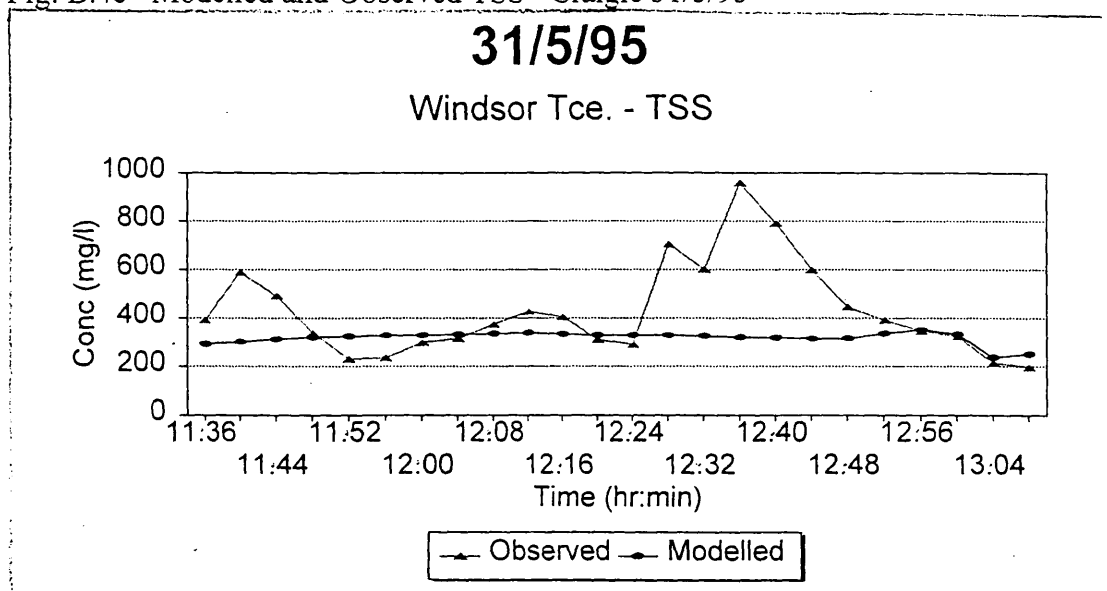


Fig. B.4d Modelled and Observed COD - Craigie 31/5/95

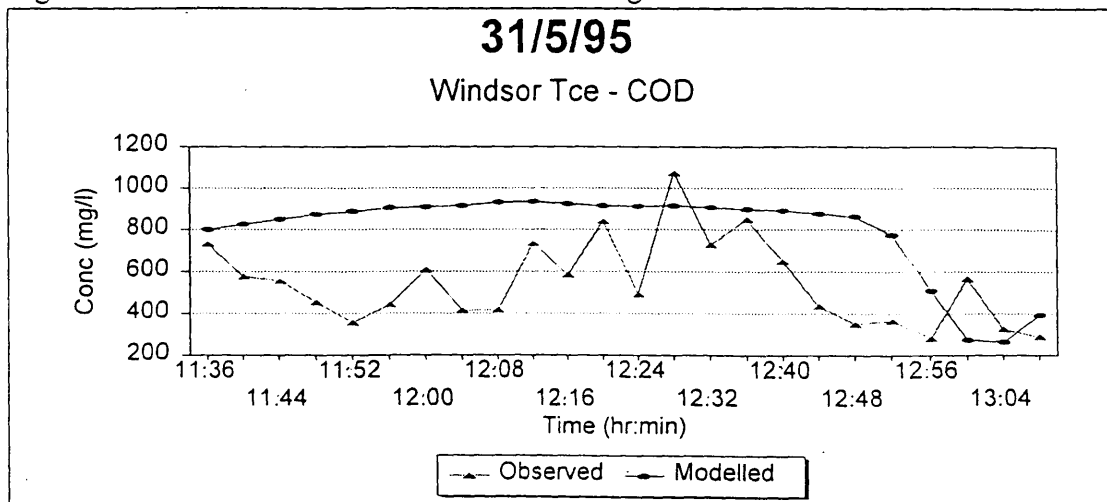
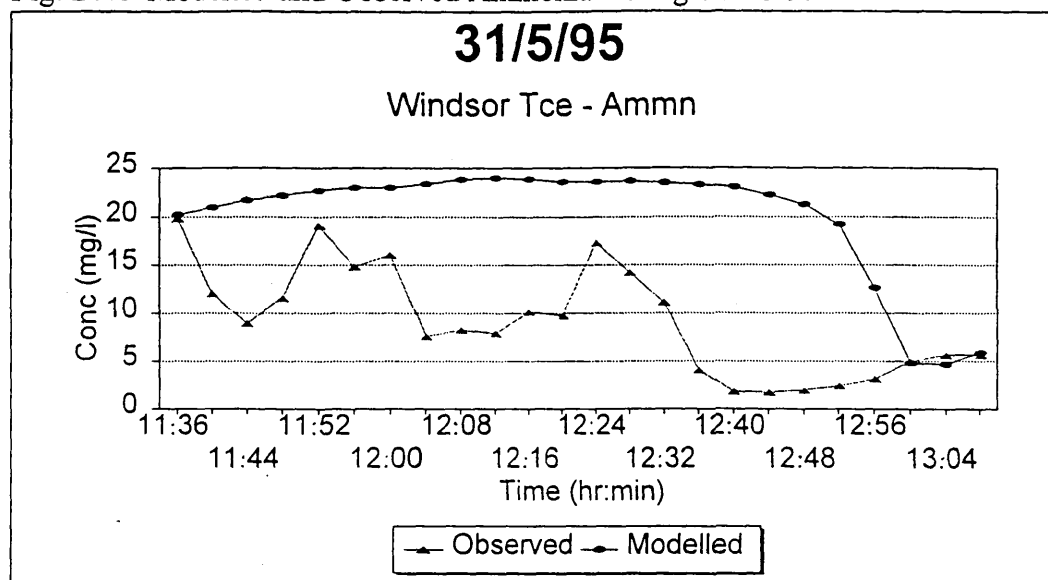


Fig. B.4e Modelled and Observed Ammonia - Craigie 31/5/95



South Inch Pumping Station

Hydraulics

This event, 31/5/95, occurred at the beginning of the data collection period and before it was possible to install a logger at the South Inch sampling site. However, due to the very good hydraulic modelling at the other locations throughout the catchment, as shown above, it is probable that the flows at the outfall would also be modelled accurately. Although this is not an ideal approach, it required to be accepted for this particular event.

Comment:

Sampling problems were encountered at the South Inch pumping station, requiring the sampler to be restarted. The problem became evident one hour and twenty minutes into the event, and only fifty minutes of subsequent data could be obtained. Consequently, the data collected from this site are from 12.50pm onward. Nevertheless, the limited data obtained proved valuable.

As can be deduced from the flow graphs in the Craigie and Bridgend subcatchments (figs. B.4a & B.3e), sampling at the South Inch pumping station began whilst the flows were subsiding. Thus the qualitative data at the pumping station were collected during the tail end of the event. Again, -QM underestimated TSS (fig. B.4f), however as time progressed the correlation between modelled and observed data improved. This trend is realistic because the contribution of sediment from the sediment bed would diminish with time. The average modelled error with respect to TSS concentration was -66% with the maximum error occurring at approximately 13.10 hrs. This error was -84%. No

analysis could be carried out with respect to the load as no flow data were available. No attempt was made to determine the error with respect to the peak as the sampling began whilst the flows were residing. Consequently, this analysis would have been meaningless.

The COD profile followed a similar trend to the TSS profile (fig.B.4g). The average error with respect to modelled COD concentration was +3%. The maximum error was -45%. The peak COD error also occurred at 13.10hrs. The average modelled error with respect to ammonia concentration was +277%. The maximum error occurring at 12:58hrs. This error was +607%.

Fig. B.4f Modelled and Observed TSS – South Inch Outfall 31/5/95

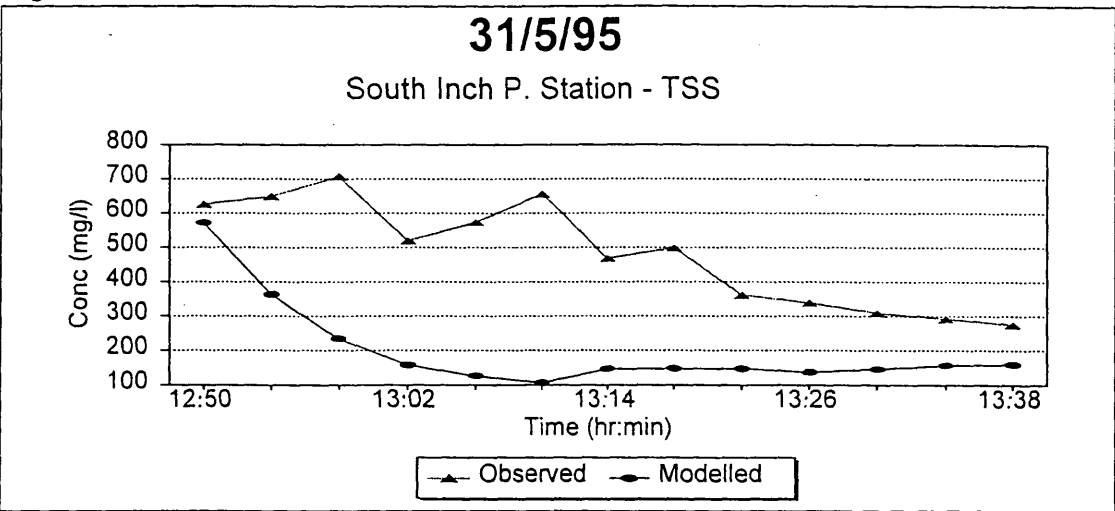


Fig. B.4g Modelled and Observed COD – South Inch Outfall 31/5/95

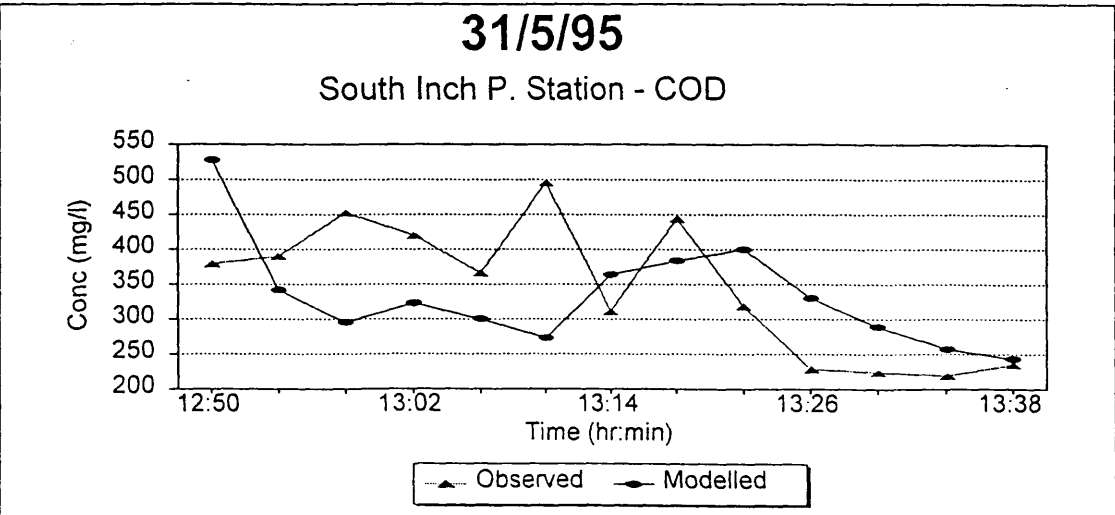


Fig. B.4h Modelled and Observed Ammonia – South Inch Outfall 31/5/95

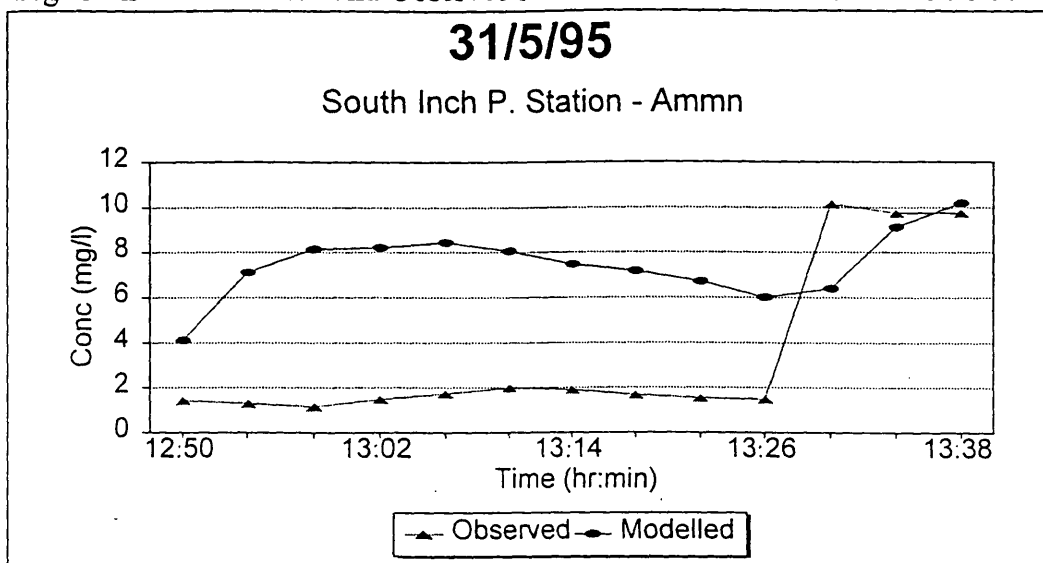


Fig B.5a Burghmuir Rainfall Profile (17/10/95)

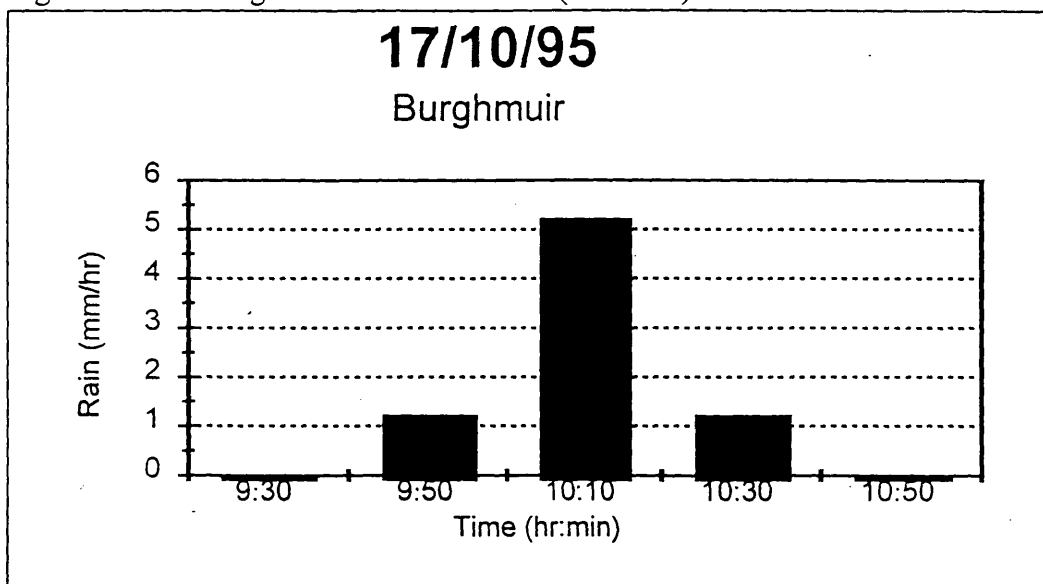


Fig B.5b Perth Grammar Rainfall Profile (17/10/95)

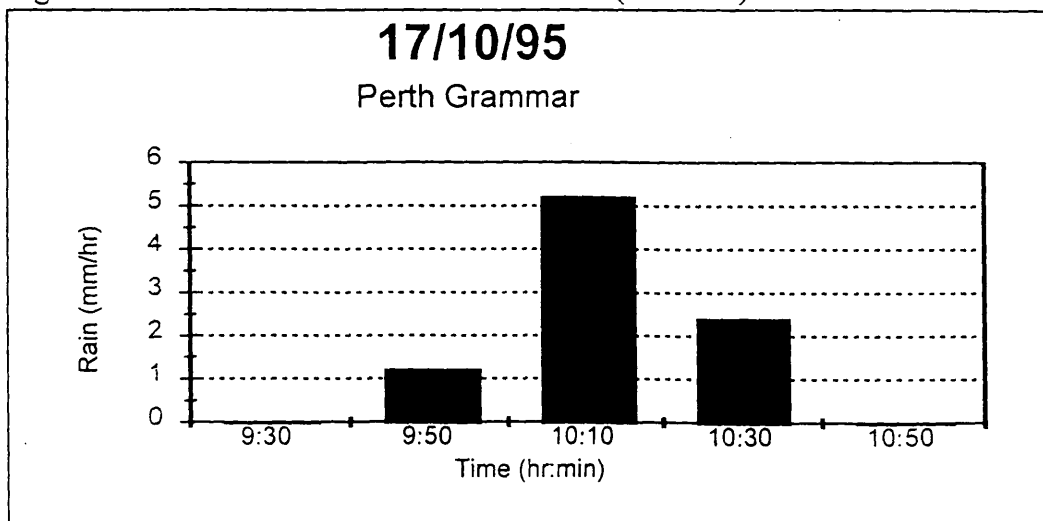


Fig B.5c Royal Murray Rainfall Profile (17/10/95)

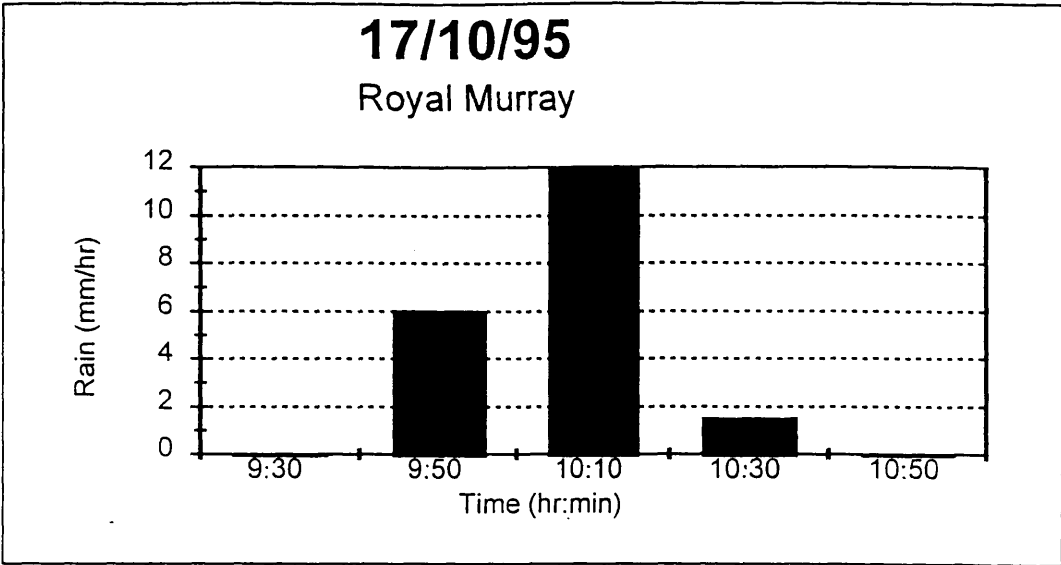


Fig B.5d Modelled Flow (North Inch – 17/10/95)

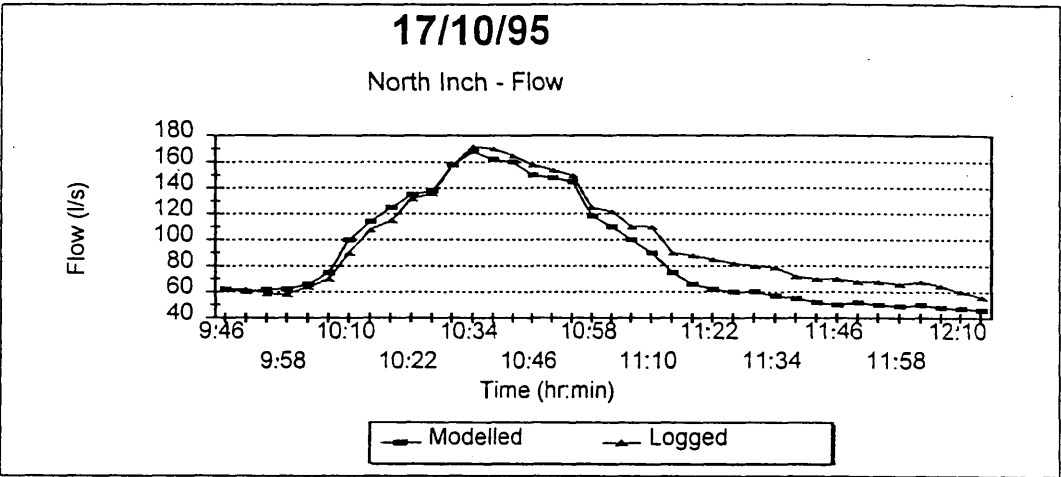
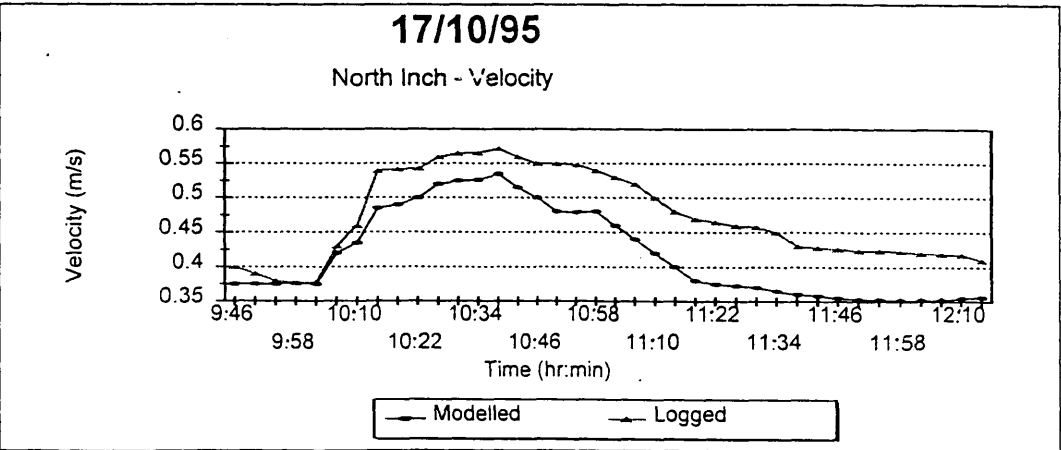


Fig B.5e Modelled Velocity (North Inch – 17/10/95)



Figures B.5d & B.5e show the hydraulic modelling at the North Inch sampling site. The modelled error with respect to total volume was -8%. The peak flow error occurred at approximately 10.34am and was -2.3%. Velocity was under predicted by 8% when the peak flow occurred. The average modelled error with

respect to velocity was -12%. Unfortunately, the rainfall event began before the sampling sites were fully operational and so sampling could only begin approximately one hour into the rainfall event. The corresponding quality plots are shown in figs B.5f, B.5g and B.5h. The peak TSS error with respect to concentration occurred at 12:14hrs and was -62%. The modelled error with respect to total TSS load was however only -26%. The observed peak occurred at 12:14pm and was 614mg/l. As there was no apparent time lag, the modelled peak was also taken at this time step. This produced a peak error of -9%. The maximum error with respect to modelled COD concentration was +205%. This occurred at 11:58hrs. The modelled error with respect to total ammonia load was +140%.

Fig B.5f Modelled and Observed TSS – North Inch 17/10/95

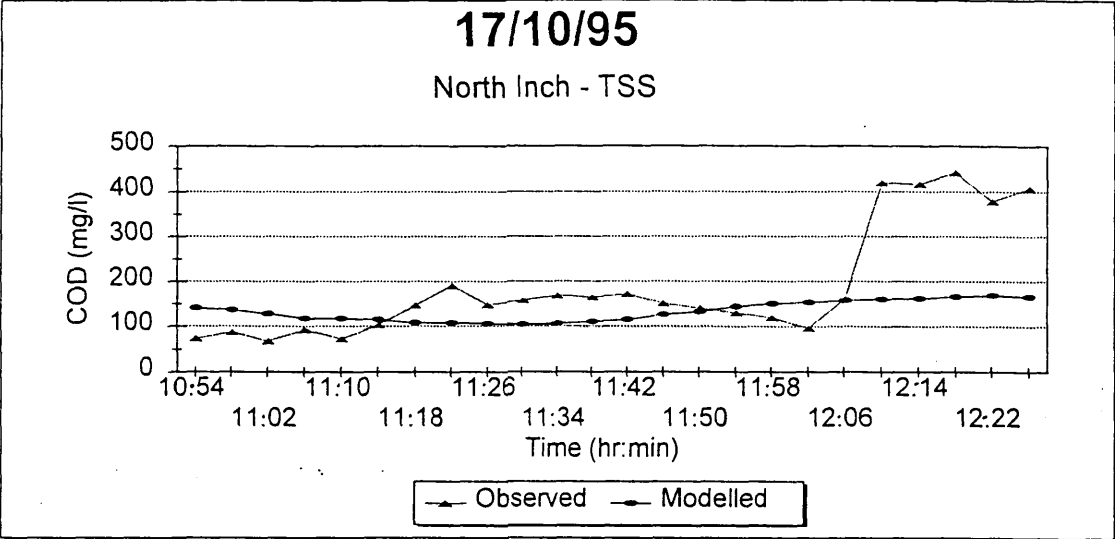


Fig. B.5g Modelled and Observed COD – North Inch 17/10/95

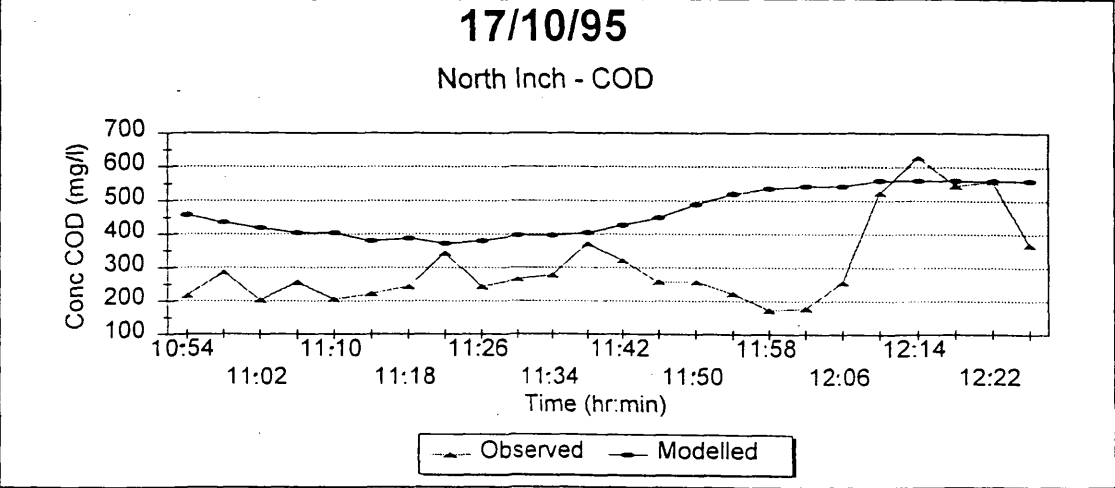


Fig. B.5h Modelled and Observed Ammonia – North Inch 17/10/95

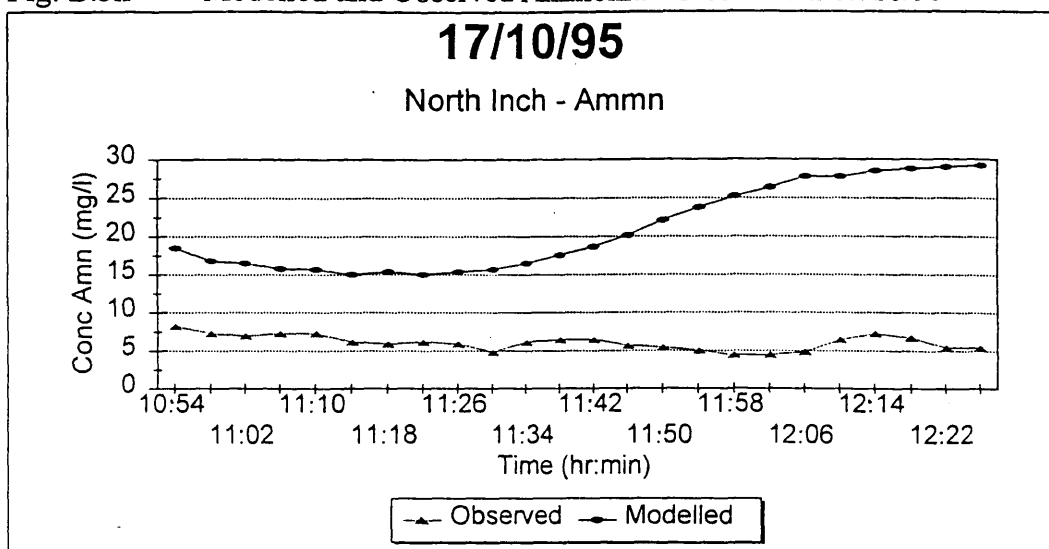


Fig. B.6a Modelled Flow (Craigie – 17/10/95)

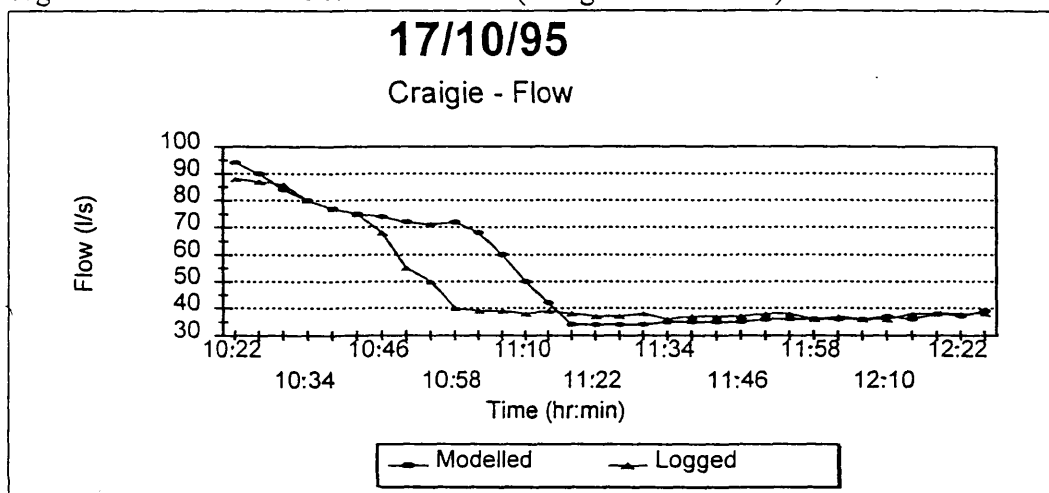
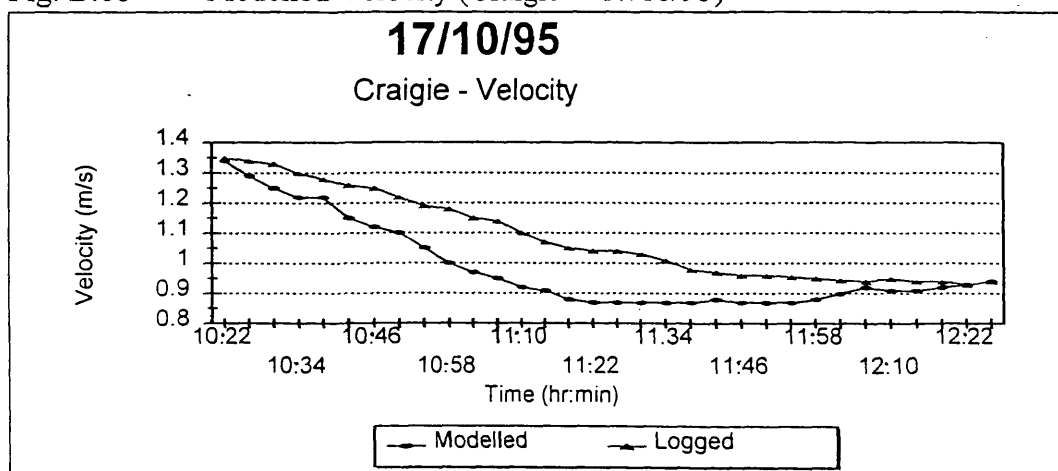


Fig. B.6b Modelled Velocity (Craigie – 17/10/95)



Figures B.6a and B.6b show the modelling of flow and velocity in the Craigie subcatchment for the 17/10/95 event. Total volume was over-predicted by +7.6%. No attempt was made to determine the peak flow error as the flows were

residing. The average modelled error with respect to velocity was -9%. Unfortunately, sampler operation at this location also began during the recession leg of the storm flows. The qualitative plots can be seen in figs B.6c, B.6d, B.6e. The general trend of the TSS profile (fig. B.6c) appears to have been modelled at this location as the modelled error with respect to total TSS load was only +8.8%. The peak error with respect to concentration was -56%. This error occurred at 11:18hrs. Total COD load was over-predicted by 83%. The peak error with respect to COD concentration was +102% at 11:26hrs (fig. B.6d). The total ammonia load was over predicted by 254%. The modelled and observed ammonia profiles can be seen in figure B.6e.

Fig. B.6c Modelled and Observed TSS – Craigie 17/10/95

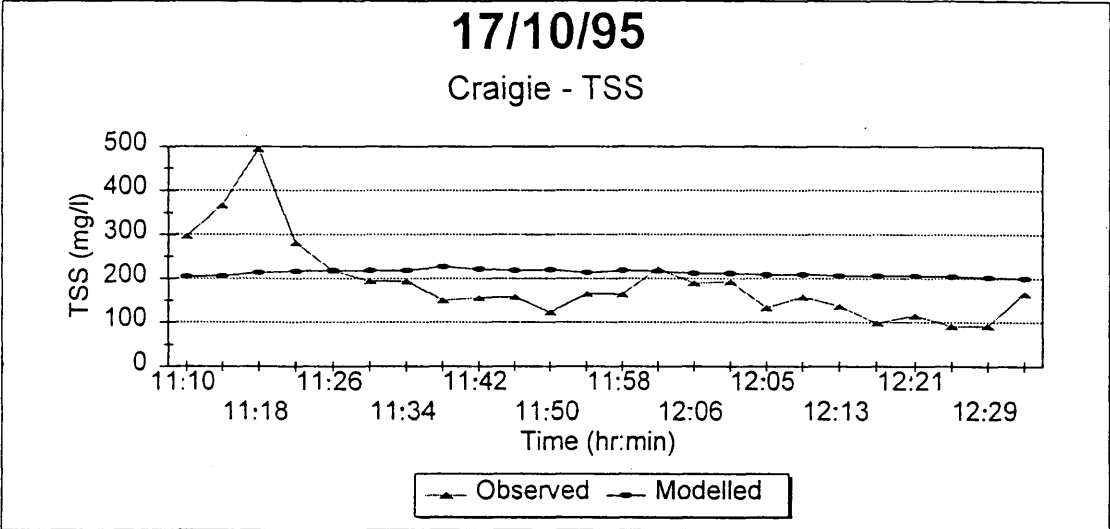


Fig. B.6d Modelled and Observed COD – Craigie 17/10/95

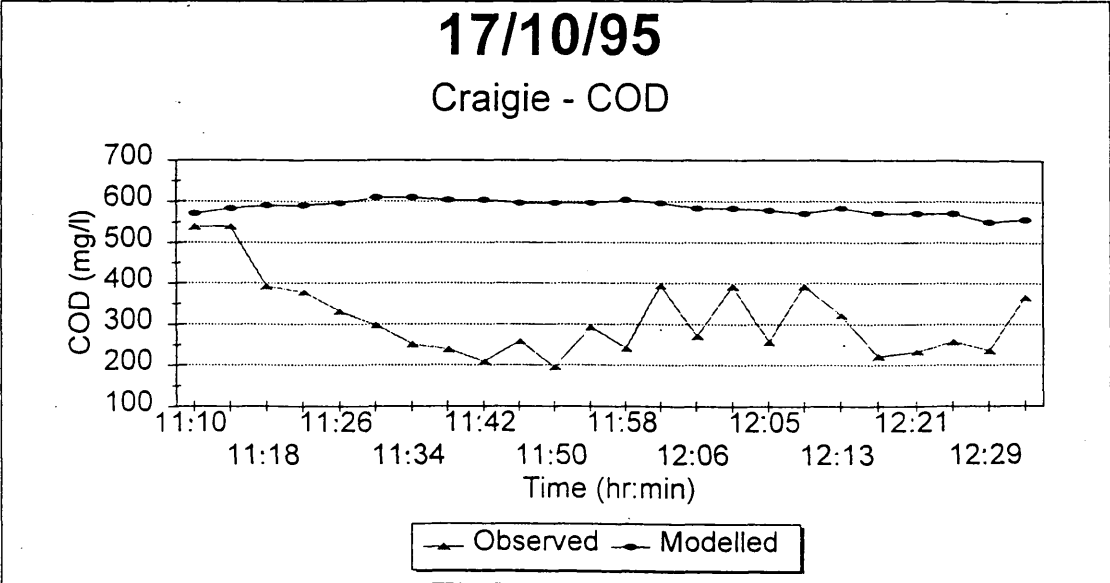


Fig. B.6e Modelled and Observed Ammonia- Craigie 17/10/95

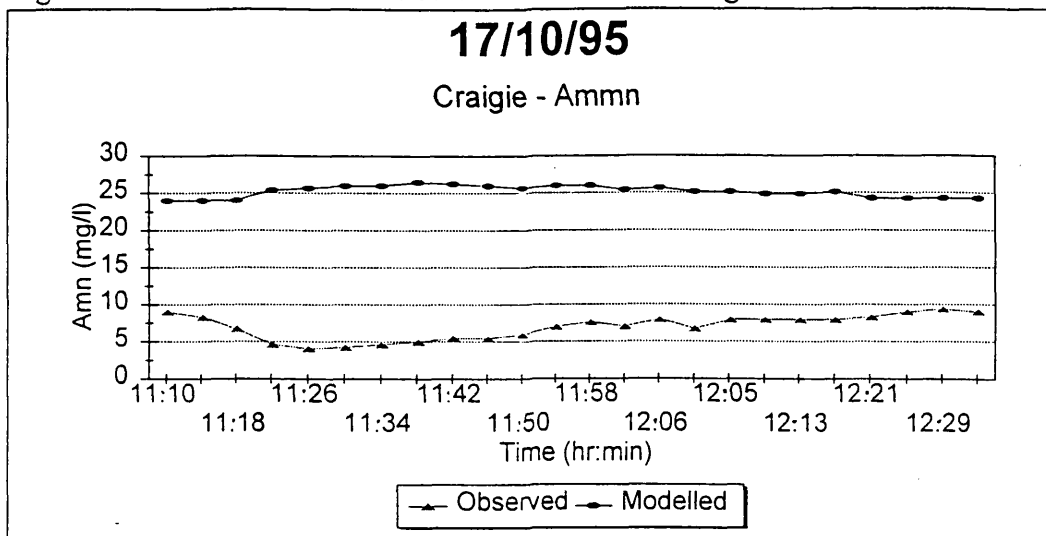


Fig B.7a Modelled Flow (South Inch Outfall – 17/10/95)

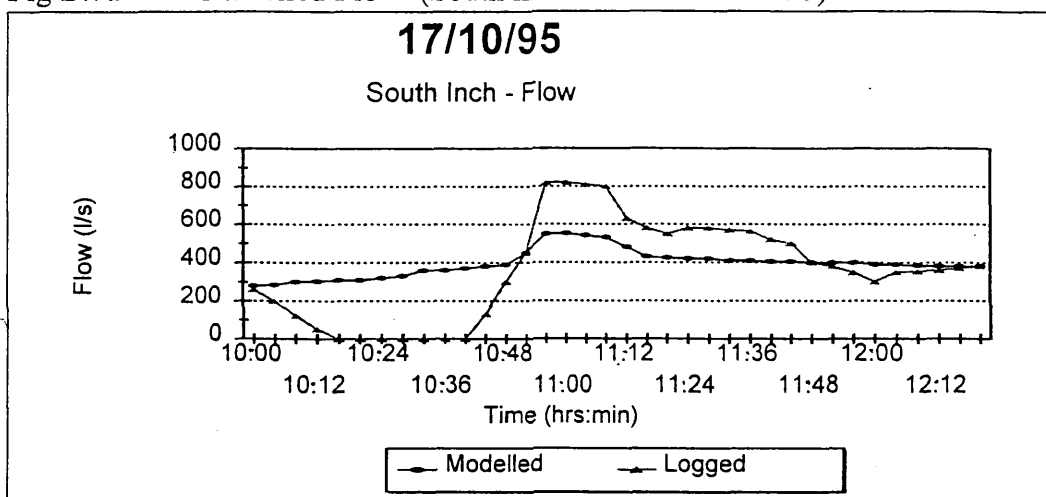
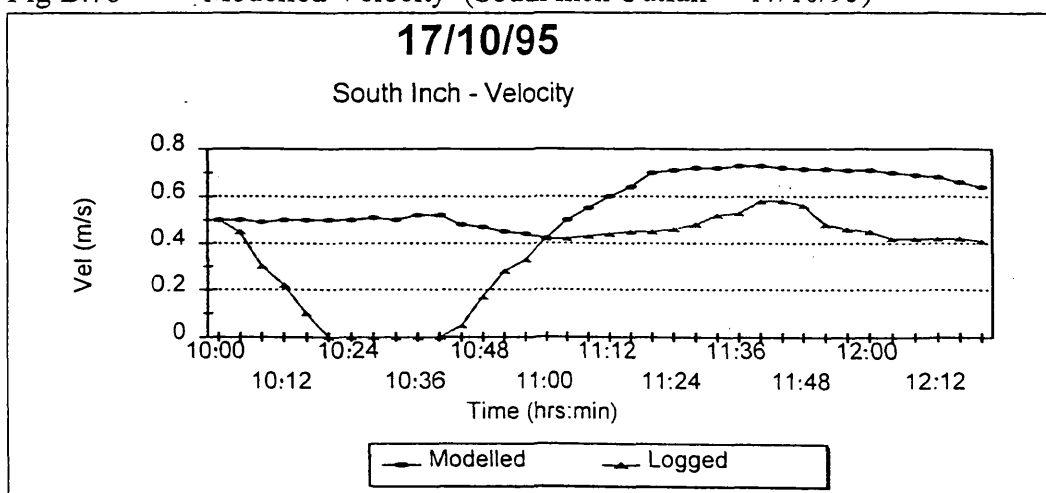


Fig B.7b Modelled Velocity (South Inch Outfall – 17/10/95)



Figures B.7a and B.7b compare modelled flow and velocity against observed data at the South Inch pumping station for the 17/10 event. Volume was over predicted by 9%, and the peak flow at approximately 10:56 hrs was under

predicted by 33%. Velocity was under predicted by 40% when the peak flow occurred. The average modelled error with respect to velocity was calculated as +232%, however, this result was skewed due to the period between 10:16 and 10:44am where no data were recorded. When these data were omitted from the analysis the average error was reduced to +40%. The poor modelling of velocity was attributed to ragging. The corresponding qualitative plots are shown in figs B.7c, B.7d and B.7e.

The total TSS load at the system outfall (South Inch Pumping Station) was under predicted by 49%. The peak modelled error with respect to TSS concentration occurred at 11:48 hrs and was -61% (fig. B.7c). As this event was quite substantial, the under prediction was likely to be a result of -QM omitting deposited sediment contributions. COD was generally modelled more accurately than TSS as the total COD load error was only -4.6% (fig. B.7d). The peak error with respect to concentration was -37% at 12:24hrs. The modelled total ammonia load was over predicted by 73% (fig. B.7e).

Fig. B.7c Modelled and Observed TSS – South Inch Outfall 17/10/95

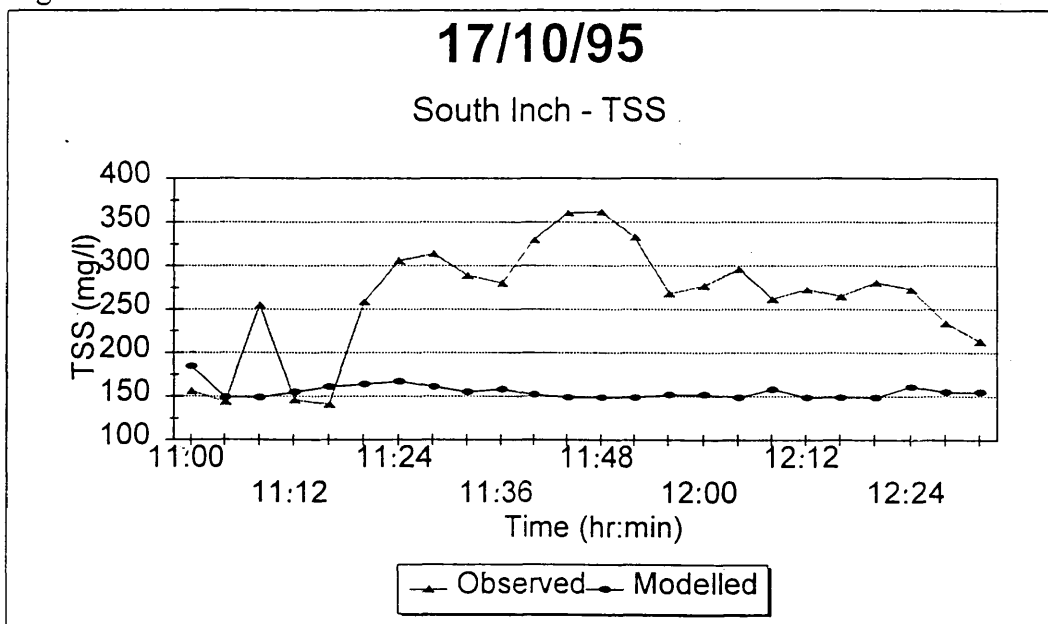


Fig. B.7d Modelled and Observed COD– South Inch Outfall 17/10/95

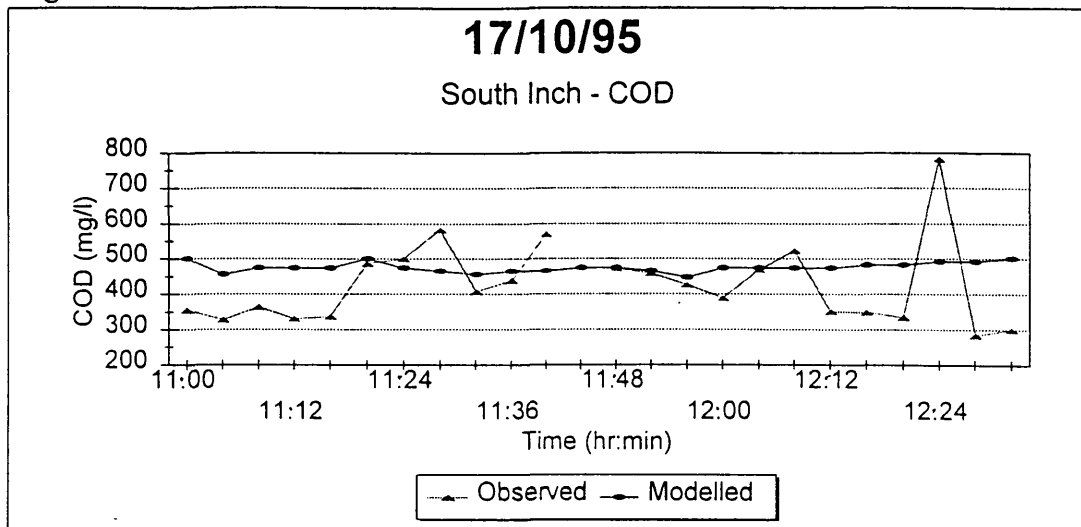
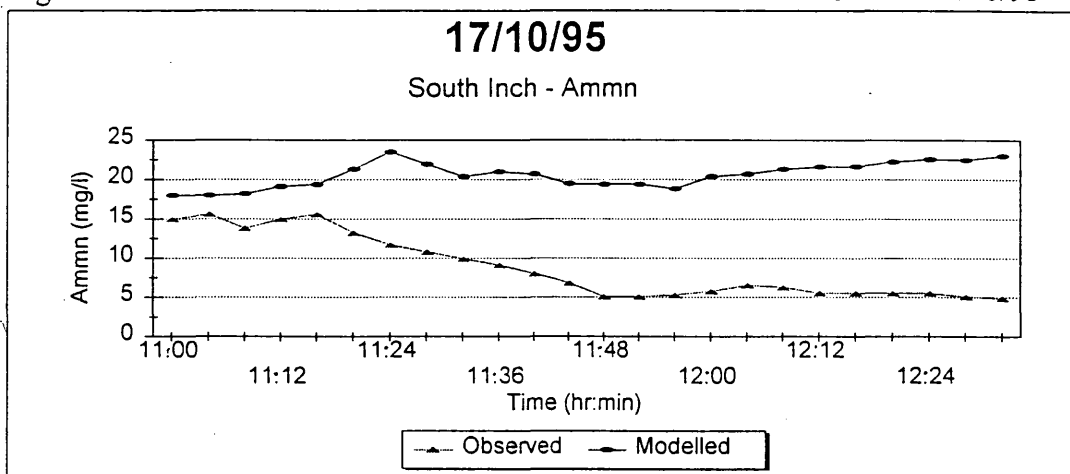


Fig. B.7e Modelled and Observed Ammonia – South Inch Outfall 17/10/95



B.4 Summary

This appendix described the storm calibration of the Hydroworks -QM model. It was shown that the hydraulics of the system were generally modelled within the guidelines defined by the WAPUG code of practise for hydraulic modelling. Nevertheless, it was apparent from the pollutographs that significant problems existed with the representation of sewer flow quality. This was particularly evident at the Willowgate sampling point in Bridgend for the 31/5 event as the total TSS load was under predicted by -82%. The error was attributed to -QM ignoring the possible contribution of sediment from deposited in-sewer sources. Ammonia was significantly over predicted for every event. The reason for this was not fully understood.

Appendix C

WTP Model Comparison:-

STOAT (Prototype), STOAT (Commercial) and GPS-X

Contents

C.1	Introduction	C3
C.2	Background	C3
C 2.1	Models 'A'	C3
C 2.2	WRc Models 1, 2, and 5	C4
C.3	Model Comparison - STOAT (Commercial) and GPS-X	C7
C.4	Summary & Conclusions - STOAT (Prototype) STOAT Commercial) and GPS-X	C9

List of Figures

C.1	STOAT ASAL1 and ASAL2 Comparison – Activated Sludge Effluent – TSS	C5
C.2	STOAT ASAL1 and ASAL2 Comparison – Activated Sludge Effluent – BOD	C6
C.3	Hydrolysis Rate Calibration – Activated Sludge Effluent - BOD	C7
C.4	GPS-X and STOAT - TSS in Activated Sludge Effluent	C8
C.5	GPS-X and STOAT - BOD in Activated Sludge Effluent	C8

Tables

C.1	Comparison of Model Errors - ASAL1 and ASAL2	C6
-----	----------------------------------------------	----

Comparison of WTP Models - STOAT (Prototype), STOAT (Commercial) and GPS-X

C.1 Introduction

The initial STOAT model used for the appraisal of the Perth sewerage system works was a prototype version of the currently available package. As discussed in chapter thirteen the prototype software was observed to over-predict the effluent BOD under periods of high hydraulic loading. An evaluation of both versions of the software was therefore undertaken to ascertain if the fully released version could provide better results than the prototype model. As a GPS-X model of the Perth system was also available the GPS-X model was also included in the comparison.

C.2 Background

A wide variety of biological models are available within the commercially available STOAT software package as shown below:-

- ASAL 1 & 1A
- ASAL 2 & 2A
- ASAL 5 & 5A
- IAWQ No. 1.
- IAWQ No. 2.

The first three models within the above list are the WRc activated sludge models. The differences between the respective packages are discussed below:-

C 2.1 Models 'A'

Models 1, 2 and 5 incorporate a simplification that de-nitrification can only take place where dissolved oxygen concentrations are zero. This means that de-nitrification only take place in deliberately created anoxic zones and that

simultaneous nitrification/de-nitrification cannot take place. The 'A' version of the models do not utilise this simplification.

C 2.2 WRc Models 1, 2, and 5

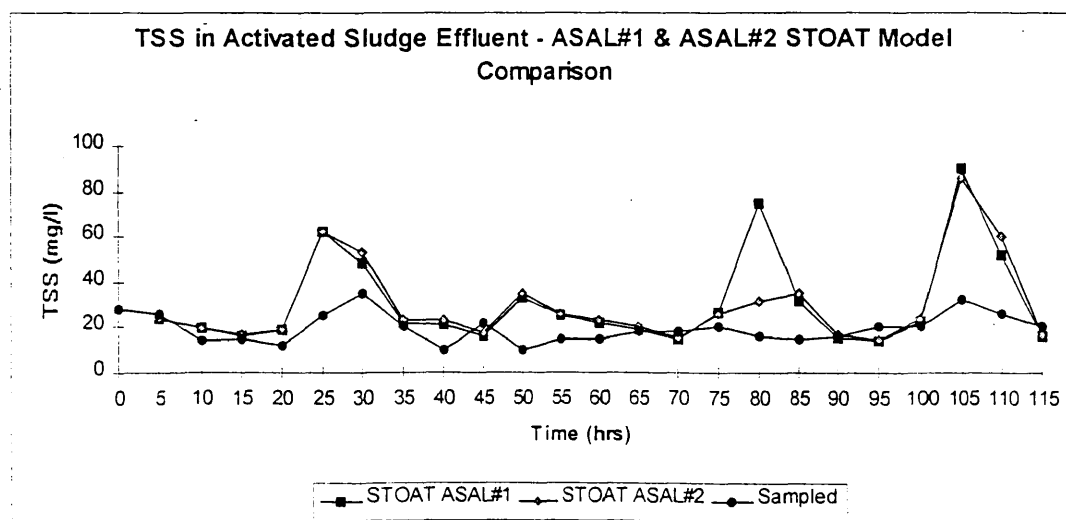
The WRc models 1, 2 and 5 are all based around the comprehensive IAWQ Activated sludge model No.1 albeit with certain simplifications.

The ASAL 1 model is the standard WRc model. This model incorporates oxidation, nitrification and de-nitrification, but does not consider the solubilisation processes of BOD. Instead a simplifying assumption is made that particulate BOD is hydrolysed rapidly. Consequently, the limitation of this model is that effluent BOD may be over-predicted when the sewage retention time is low. The reason being that insufficient time is available to treat the 'assumed' immediate solubilisation of particulate BOD. This model is therefore recommended when the sewage residence time within the reactor is greater than 2-4 hrs. When the retention time is less than 2 to 4 hrs the ASAL 2 model is recommended. The fundamental difference between the ASAL 2 model and the ASAL 1 model is that ASAL 2 gives greater consideration to the hydrolysis processes. Consequently, within ASAL2 solubilisation occurs over time instead of immediately. The ASAL 5 includes a simple model for biological phosphorous removal. The models assumes that phosphorous is removed in proportion to the biomass growth. Although this is a general simplification of the biological processes which take place it is believed to be appropriate with respect to *engineering* predictions of phosphorous removal (STOAT User Guide, 1994)¹. As nitrification and de-nitrification process do not take place at the Sleepless Inch WTP in Perth only ASAL models one and two were utilised in the comparison. The prototype STOAT model which was partially constructed in 1993 by NoSWA (in collaboration with WRc) was constructed using ASAL1. An initial comparison was carried out in-order to ensure no fundamental differences existed between the effluent quality predicted by the ASAL1 model (prototype version) and the ASAL1 model (fully released version) whilst using the same

¹ STOAT User Guide, 1994 STOAT User Guide, Documentation from WRc plc 1994

calibration parameters. It was found that no differences existed, however a difference in settlement tank modelling procedures required that a feed point (distance to bottom of baffle) needed to be defined when using the commercial available package. This was not a requirement of the prototype software. Unfortunately, the original drawings of the WTP could not be located and this data could not be measured on site as the tanks were in use. Consequently, the feed point was used as a calibration parameter. It was found that suitable results were gained when the feed point was placed at 1.68m from the top water level. As published data states that the depth of baffles are typically 1-2.5 m for a circular tank with central feed (Metcalf and Eddy, 1991)² this fell within the suitable range. No differences were noted between the outputs of the two ASAL1 models. As discussed in chapter thirteen, the prototype model substantially over-predicted BOD₅ during periods of high hydraulic loading. This, as previously discussed, was a consequence of the underlying assumption that the particulate BOD is hydrolysed immediately. As the ASAL2 model within the commercially available package gives greater consideration to the hydrolysis processes a comparative test was carried out to ascertain whether more accurate results would be obtained. The results are shown in figures C.1 and C.2.

Fig. C.1 STOAT ASAL1 and ASAL2 Comparison – Activated Sludge Effluent – TSS



² Metcalf and Eddy. (1991) Treatment, Disposal and Reuse. Third Edition. Publishers: McGraw Hill International Editions. (Civil Engineering Series).

Fig.C.2 STOAT ASAL1 and ASAL2 Comparison – Activated Sludge Effluent – BOD

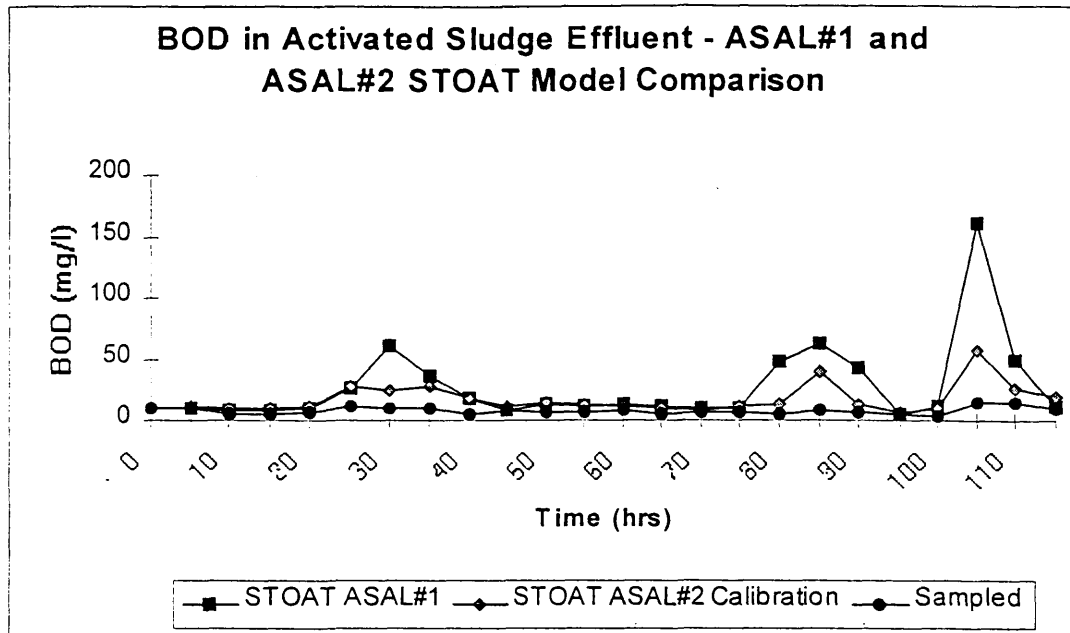


Table C.1 provides a comparison of the modelling errors depicted by figures C.1 and C.2.

Table C.1 Comparison of Model Errors - ASAL1 and ASAL2

Activated Sludge Effluent	TSS	BOD
Average Error – ASAL1	+71%	+204%
Average Error – ASAL2	+74%	+162%
Peak Error - ASAL1	+57.1%	+966.6%
Peak Error – ASAL2	+57.1%	+500%

It can be seen from table C.1 and from figure C.1 that TSS were modelled to a similar degree of accuracy irrespective of whichever package was used. Although, it can be seen from figure C.1 that ASAL2 provided better results at hour 80. It was apparent that the modelling of BOD was still subject to problems (fig. C.2), although the peak over-prediction was reduced from +966% to +500%. This therefore suggested that the default hydrolysis rate chosen within the ASAL 2 model was better, although still too high for the Sleepless Inch WTP. Further

calibration of the hydrolysis parameter showed that the outputs could be further improved (fig. C.3)

Fig. C.3 Hydrolysis Rate Calibration – Activated Sludge Effluent - BOD

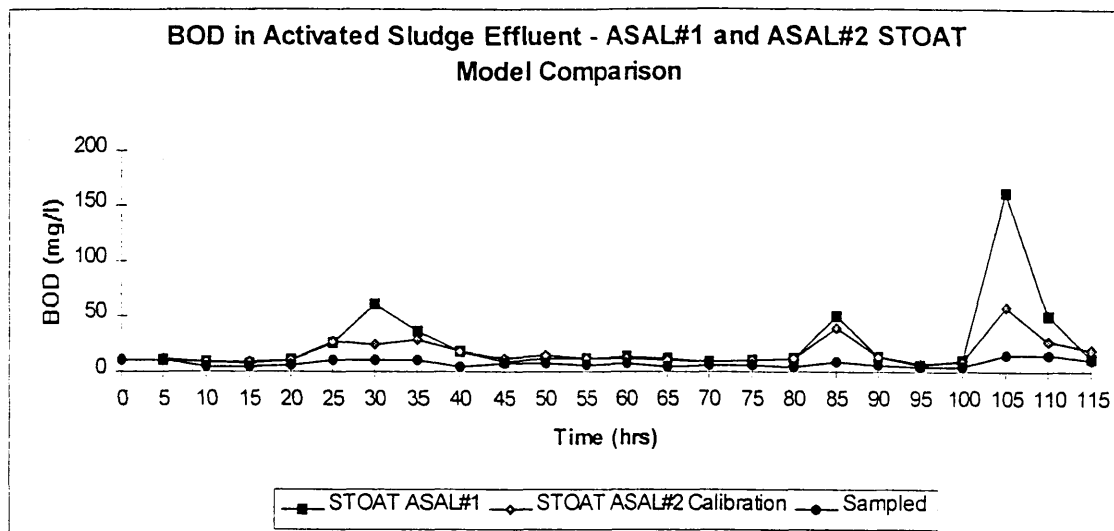


Fig. C.3 shows that calibration of the hydrolysis parameter within the ASAL2 model provided substantially better results for BOD. The average modelled error was reduced from +162% to +127% and the peak error at hour 105 was reduced from +500% to +280%.

C.3 Model Comparison - STOAT (Commercial) and GPS-X

A comparison was carried out between the WRc ASAL2 model and the GPS-X model. It should be noted that no direct comparison could be made between the kinetic constants due to differences in the way the respective packages operate e.g. GPSX's stoichiometric data are based upon 'mg/l' as COD equivalent, with reported effluent BOD being calculated as degradable COD multiplied by a conversion factor. In STOAT, it is somewhat different. The stoichiometric data are measured as mg/l of solids (not mg/l of COD) and BOD is calculated as BOD and not biodegradable COD.

Comparative plots can be seen overleaf comparing WTP effluent using ASAL2 and GPS-X.

Fig. C.4 GPS-X and STOAT - TSS in Activated Sludge Effluent

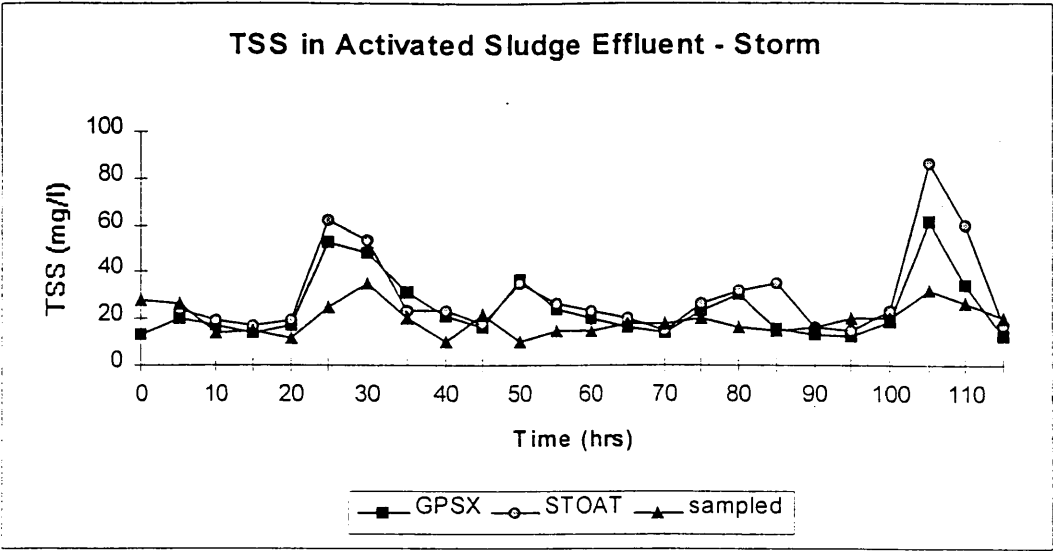


Fig.C.5 GPS-X and STOAT - BOD in Activated Sludge Effluent

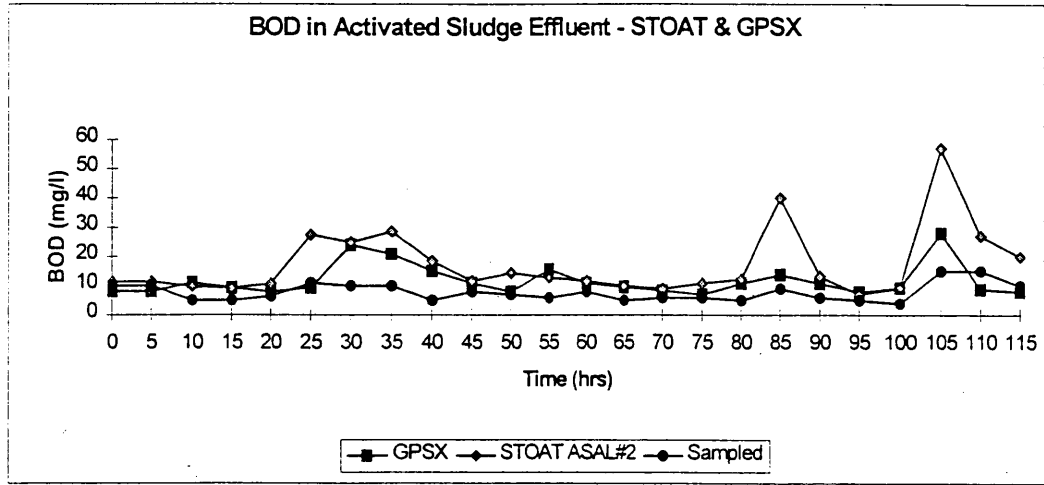


Figure C.4 shows that there is a marginal difference between TSS predictions using the GPS-X and STOAT ASAL2 models. The average modelled error using STOAT ASAL2 was +74% with a peak error of 54%. The average modelled error using GPS-X was 32% with the peak error being 51.4%. Both models have over-predicted the TSS effluent although it can be seen that GPS-X over-predicted to a lesser degree.

With respect to BOD (fig. C.5), the average modelled error using GPS-X was +69%, compared to the +127% error using STOAT ASAL2. The gross BOD over-prediction (+280%) which occurred at time step 105 hours using STOAT did not occur using GPS-X. The error obtained using GPS-X was +114.2%.

should be noted however that this does not mean GPS-X is automatically a better model, but that further calibration of the STOAT model is required. This was not carried out as no data were available to determine a suitable calibration value for the hydrolysis parameter. Consequently, improvements to the modelled results would therefore constitute a force fitting of the model.

C.4 Summary & Conclusions - STOAT (Prototype) STOAT Commercial) and GPS-X

The fundamental difference between the two WRc STOAT is that the ASAL1 assumed a very rapid rate of hydrolysis. The consequence of this simplification was that the model over predicted BOD during storms. The ASAL2 model gave greater consideration to the hydrolysis processes, however BOD prediction were still over-predicted, albeit less so. It was noted that further calibrations to the hydrolysis parameter could improve the modelled outputs.

The GPS-X model was constructed based on the calibrations used to calibrate the ASAL1 STOAT model. No significant problems were found during calibration. TSS were modelled to a similar degree of accuracy in STOAT and GPS-X however, BOD predictions from GPS-X were found to be more accurate than both STOAT models (ASAL1 and ASAL2).

It was observed that GPS-X provided a substantially better representation of BOD than the prototype version of the STOAT software, however, as the full version of STOAT could be further calibrated it was not concluded that GPS-X was better than this version of the WRc software. It was concluded however that GPS-X and the full version of the STOAT software were both very comprehensive packages with neither offering any significant advantage over the other.

Appendix D

Sewer Flow Quality Modelling - Supporting Graphs

List of Figures

D.1	DWF at System Outfall (MOSQUITO)	D4
D.2	TSS at System Outfall (MOSQUITO)	D4
D.3	COD at System Outfall (MOSQUITO)	D4
D.4	Ammonia at System Outfall (MOSQUITO)	D4
D.5	DWF in Bridgend Sub-catchment (-QM)	D5
D.6	Ammonia in Bridgend Sub-catchment (-QM)	D5
D.7	TSS in Bridgend Sub-Catchment (-QM)	D5
D.8	COD in Bridgend Sub-Catchment (-QM)	D5
D.9	DWF in Craigie Sub-catchment(-QM)	D6
D.10	Ammonia in Craigie Sub-catchment (-QM)	D6
D.11	TSS in Craigie Sub-catchment (-QM)	D6
D.12	COD in Craigie Sub-catchment (-QM)	D6
D.13	DWF in Muirton Sub-catchment (-QM)	D7
D.14	Ammonia in Muirton Sub-catchment (-QM)	D7
D.15	TSS in Muirton Sub-catchment (-QM)	D7
D.16	COD in Muirton Sub-catchment (-QM)	D7
D.17	DWF at System Outfall (-QM)	D8
D.18	TSS at System Outfall (-QM)	D8
D.19	Ammonia at System Outfall (-QM)	D9
D.20	COD at System Outfall (-QM)	D9
D.21	Modelled and Observed BOD – North Muirton (DWF)	D10
D.22	Modelled and Observed BOD – Bridgend (DWF)	D10
D.23	Modelled and Observed BOD – Tullton (DWF)	D10
D.24	Modelled and Observed BOD – System Outfall (DWF)	D11
D.25	Modelled and Observed Flow – Bridgend (24/8/95)	D11
D.26	Modelled and Observed BOD - Bridgend (24/8/95)	D11
D.27	Modelled and Observed Flow – South Inch Outfall (24/8/95)	D12
D.28	Modelled and Observed BOD – South Inch Outfall (24/8/95)	D12
D.29	Modelled and Observed Flow – South Inch Outfall (29/8/95)	D12
D.30	Modelled and Observed BOD – South Inch Outfall (29/8/95)	D13

List of Figures

D.31	Modelled and Observed Flow – Bridgend (31/5/95)	D13
D.32	Modelled and Observed BOD – Bridgend (31/5/95)	D13
D.33	Modelled and Observed Flow – Craigie (31/5/95)	D14
D.34	Modelled and Observed BOD – Craigie (31/5/95)	D14
D.35	Modelled and Observed BOD – South Inch Outfall (31/5/95)	D14
D.36	Initialisation Period Sensitivity Testing	D15
D.37	ADWP Sensitivity Test	D15
D.38	Maximum Possible ADWP Analysis (TSS)	D15
D.39	Maximum Possible ADWP Analysis (COD)	D16
D.40	“Infiltration” and “No Infiltration” Model Comparison – Flow	D16
D.41	“Infiltration” and “No Infiltration” Model Comparison – TSS	D16
D.42	“Infiltration” and “No Infiltration” Model Comparison – COD	D17
D.43	Diameter Sensitivity Analysis - TSS (Bridgend - DWF)	D17
D.44	Diameter Sensitivity Analysis - COD (Bridgend - DWF)	D17
D.45	Diameter Sensitivity Analysis - TSS (Outfall - DWF)	D18
D.46	Diameter Sensitivity Analysis - COD (Outfall – DWF)	D18
D.47	Diameter Analysis - TSS (Storm)	D18
D.48	Diameter Analysis – COD Storm	D18
D.49	Specific Gravity Sensitivity Analysis - TSS (Bridgend - DWF)	D19
D.50	Specific Gravity Sensitivity Analysis - COD (Bridgend - DWF)	D19
D.51	Specific Gravity Sensitivity Analysis - TSS (Outfall – DWF)	D19
D.52	Specific Gravity Sensitivity Analysis - COD (Outfall – DWF)	D20
D.53	S.G. Analysis - TSS (Storm)	D20
D.54	S.G. Analysis - COD (Storm)	D20
D.55	Additional S.G Analysis – TSS (Outfall – DWF)	D20
D.56	Additional S.G Analysis – COD (Outfall – DWF)	D21
D.57	Additional S.G. Analysis - TSS (Storm)	D21
D.58	Additional S.G. Analysis - COD (Storm)	D21

Fig. D.1 DWF at System Outfall (MOSQITO)

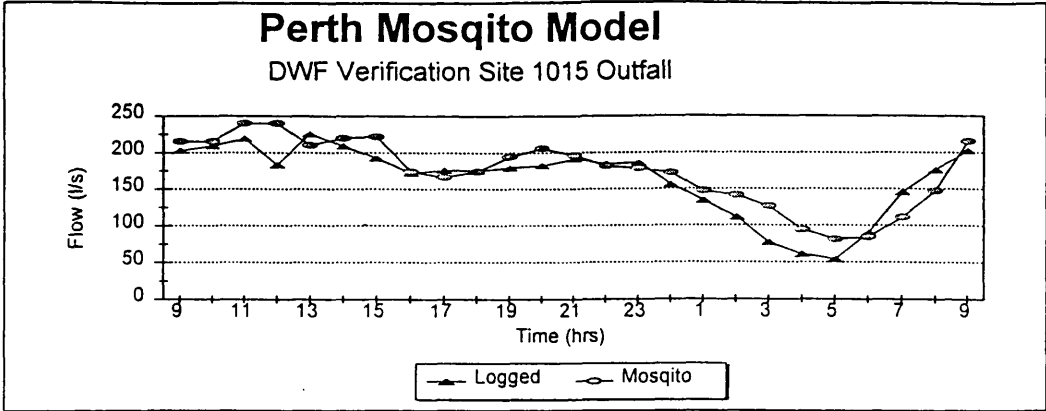


Fig D.2 TSS at System Outfall (MOSQITO)

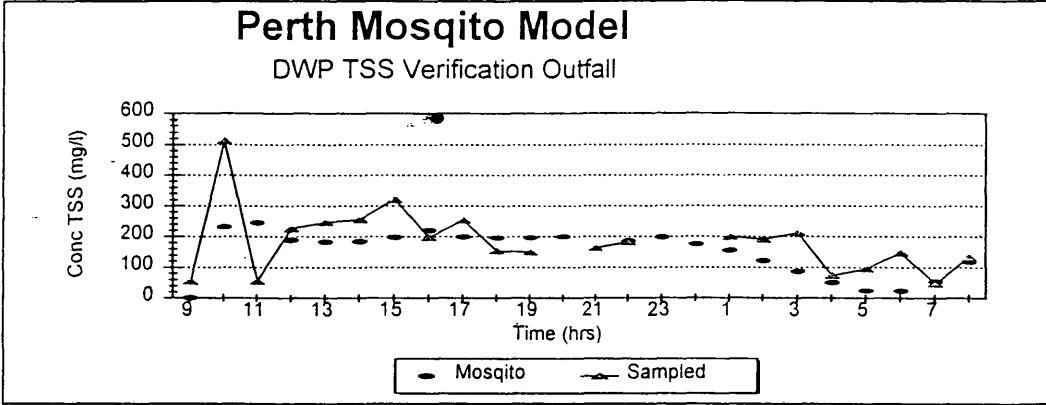


Fig. D.3 COD at System Outfall (MOSQITO)

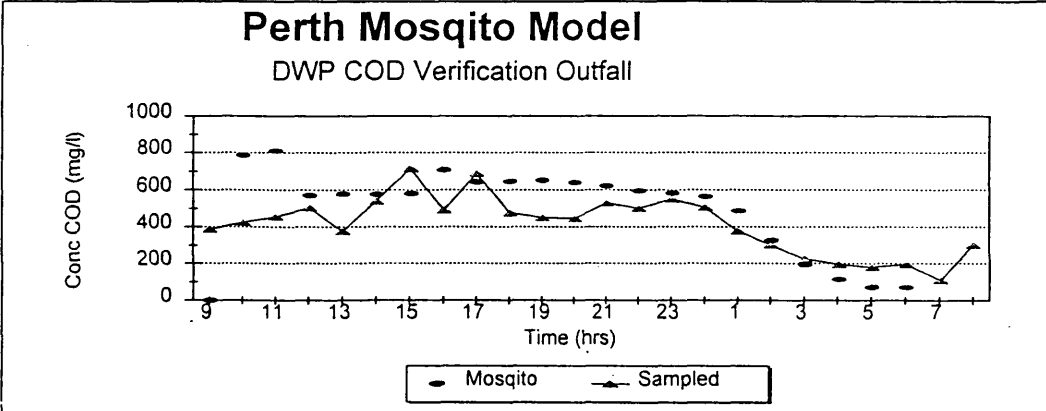


Fig. D.4 Ammonia at System Outfall (MOSQITO)

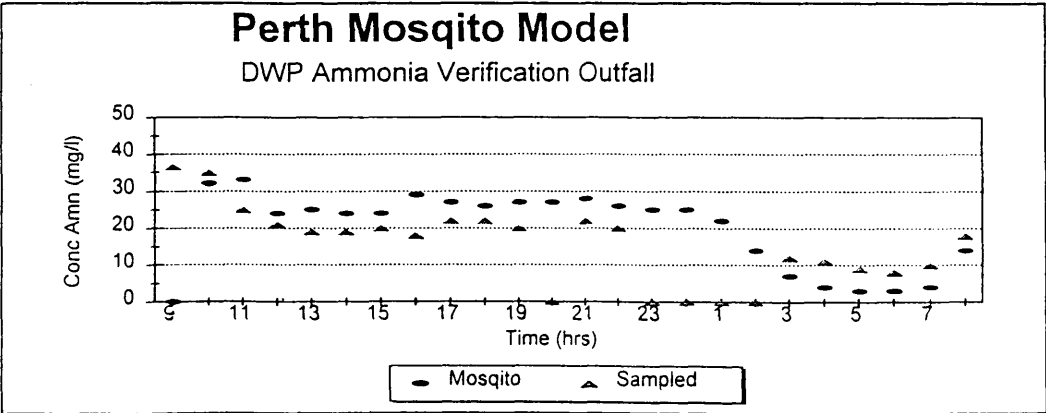


Fig D.5

DWF in Bridgend Sub-catchment (-QM)

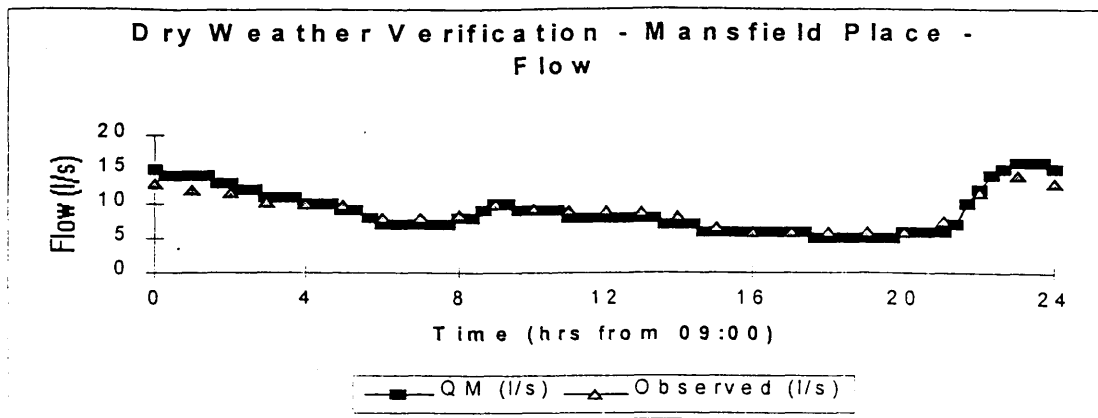


Fig. D.6

Ammonia in Bridgend Sub-catchment (-QM)

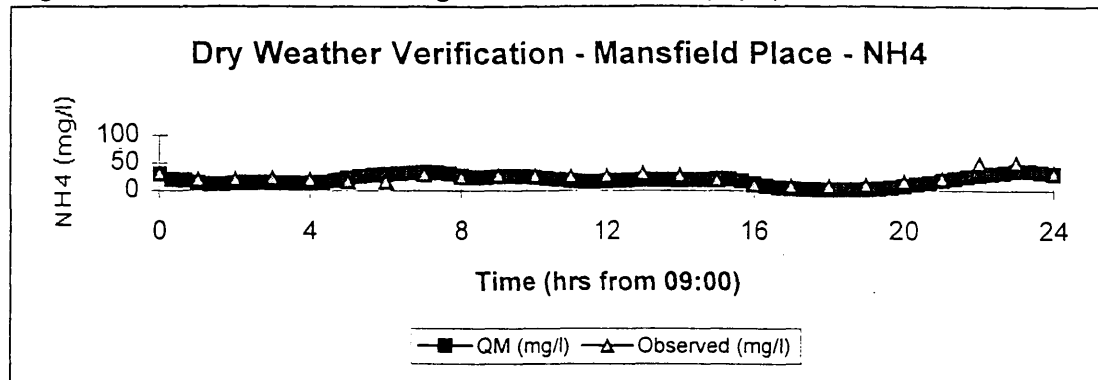


Fig. D.7

TSS in Bridgend Sub-Catchment (-QM)

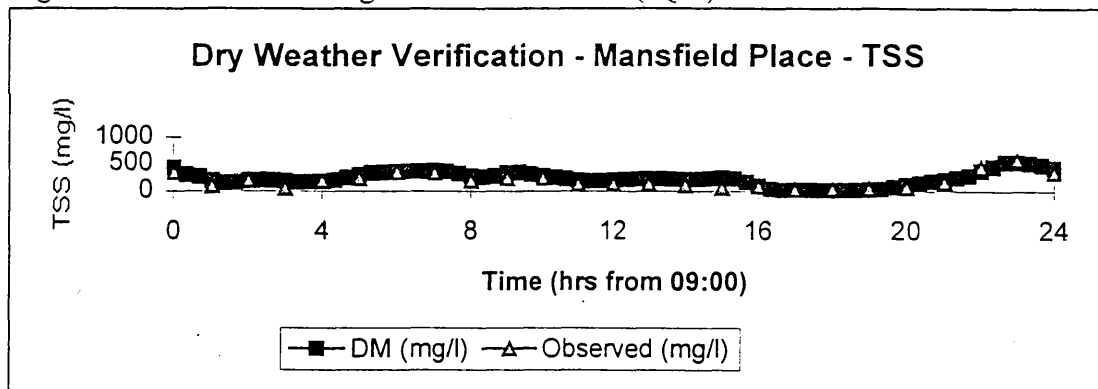


Fig. D.8

COD in Bridgend Sub-Catchment (-QM)

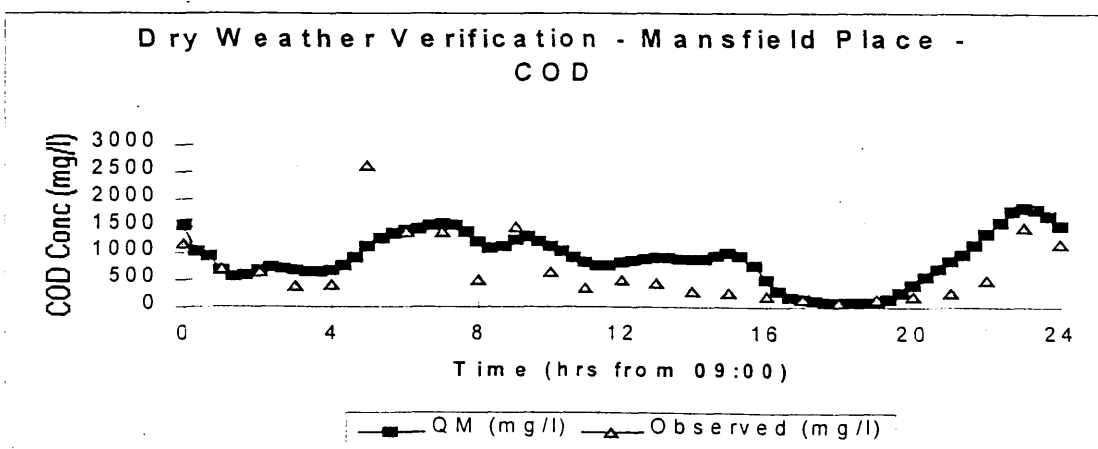


Fig. D.9 DWF in Craigie Sub-catchment(-QM)

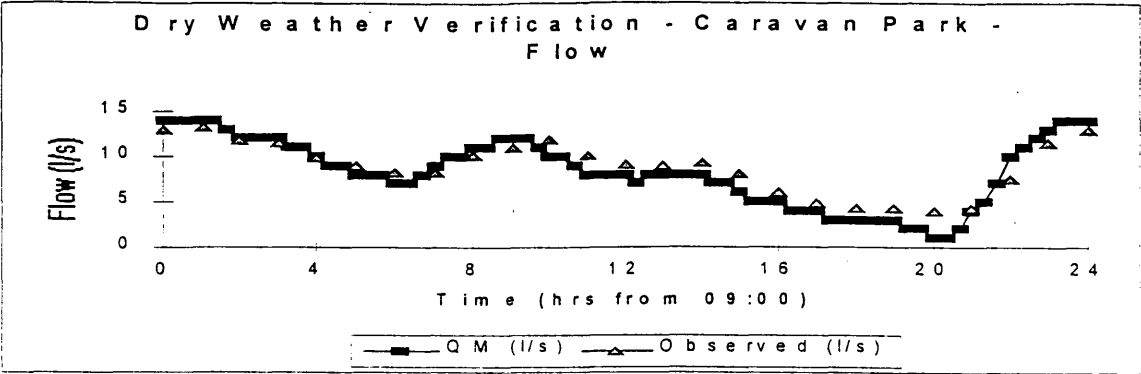


Fig. D.10 Ammonia in Craigie Sub-catchment (-QM)

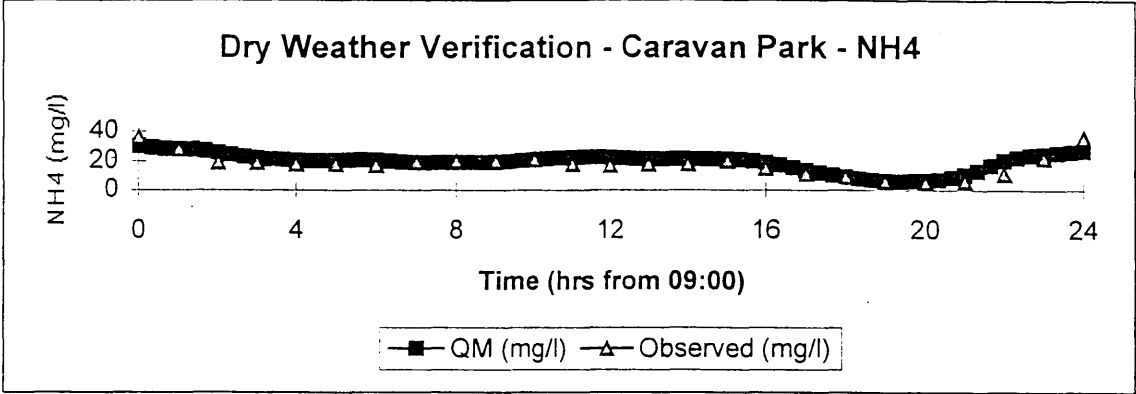


Fig. D.11 TSS in Craigie Sub-catchment (-QM)

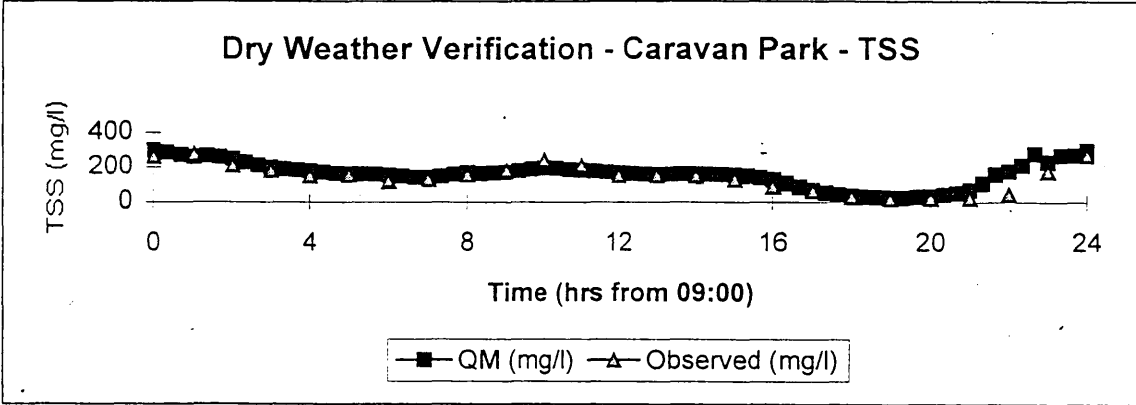


Fig. D.12 COD in Craigie Sub-catchment (-QM)

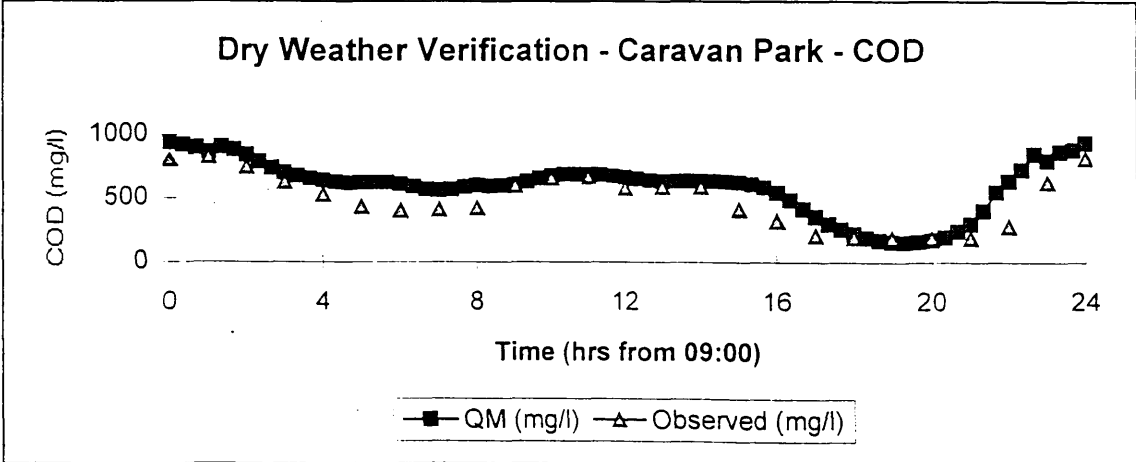


Fig D.13 DWF in Muirton Sub-catchment (-QM)

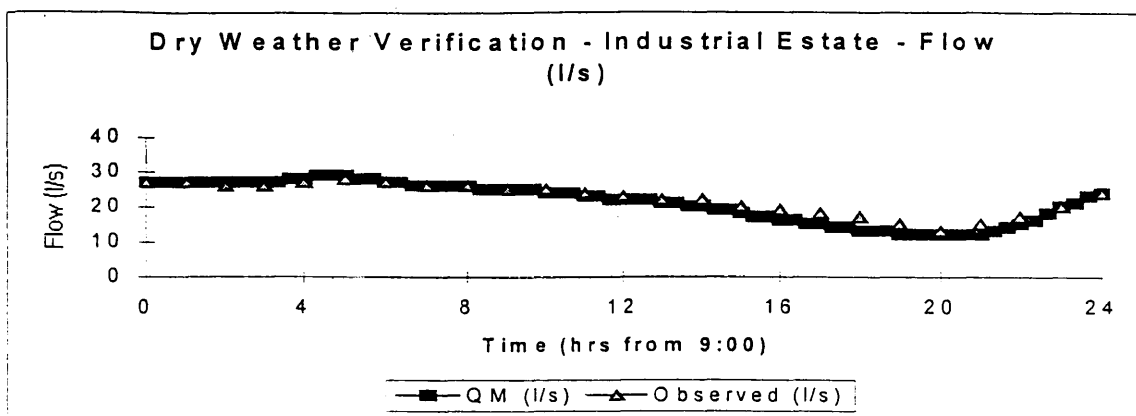


Fig D.14 Ammonia in Muirton Sub-catchment (-QM)

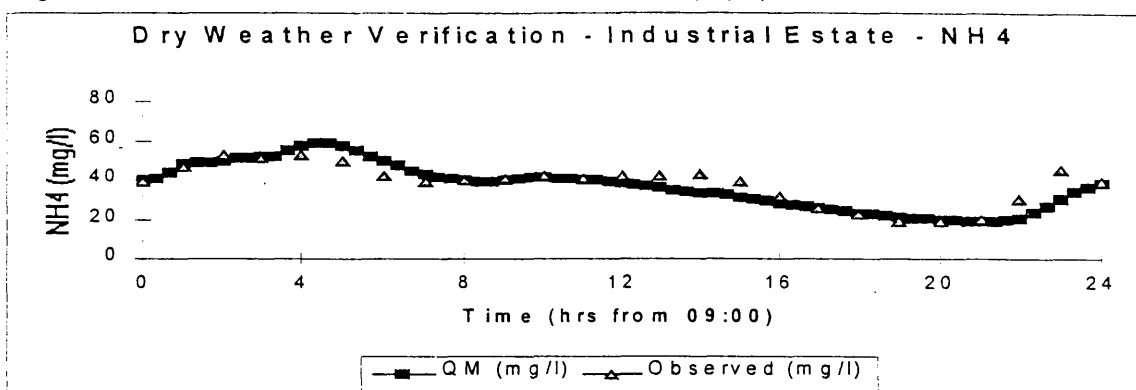


Fig D.15 TSS in Muirton Sub-catchment (-QM)

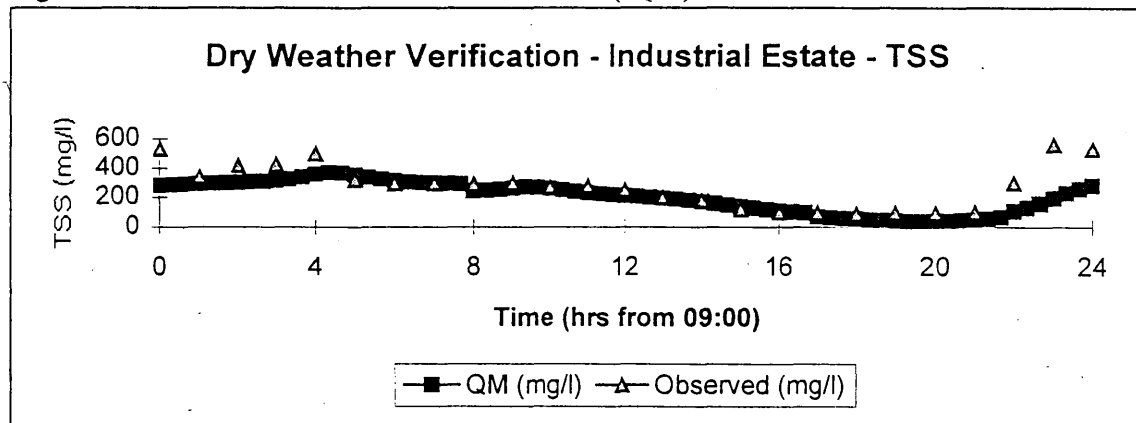


Fig D.16 COD in Muirton Sub-catchment (-QM)

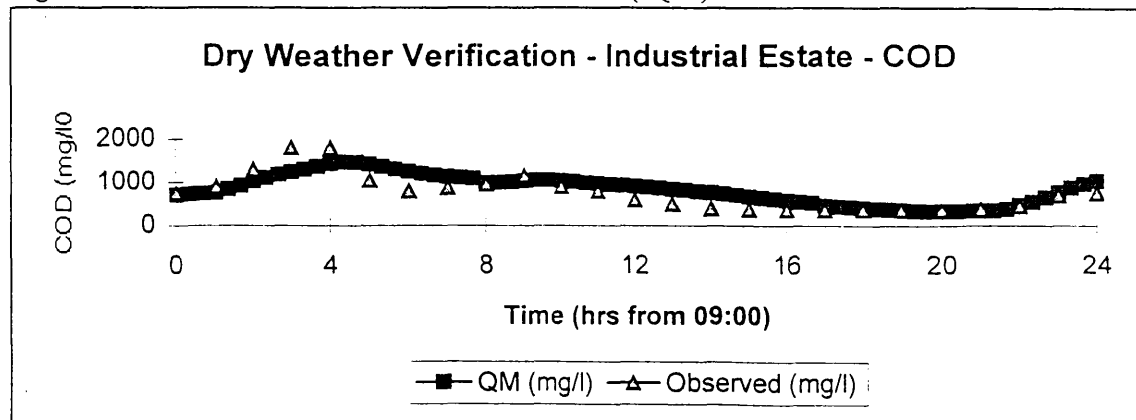


Fig. D.17 DWF at System Outfall (-QM)

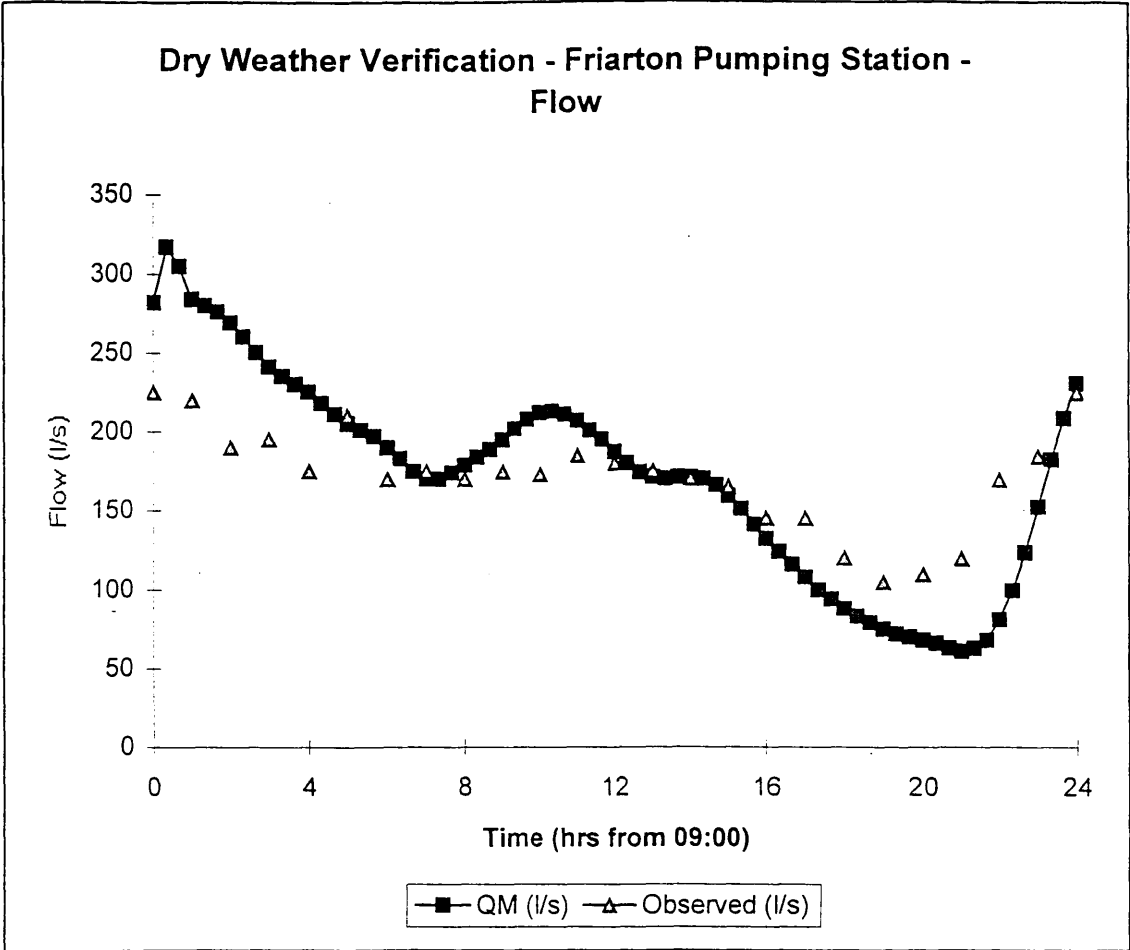


Fig. D.18 Ammonia at System Outfall (-QM)

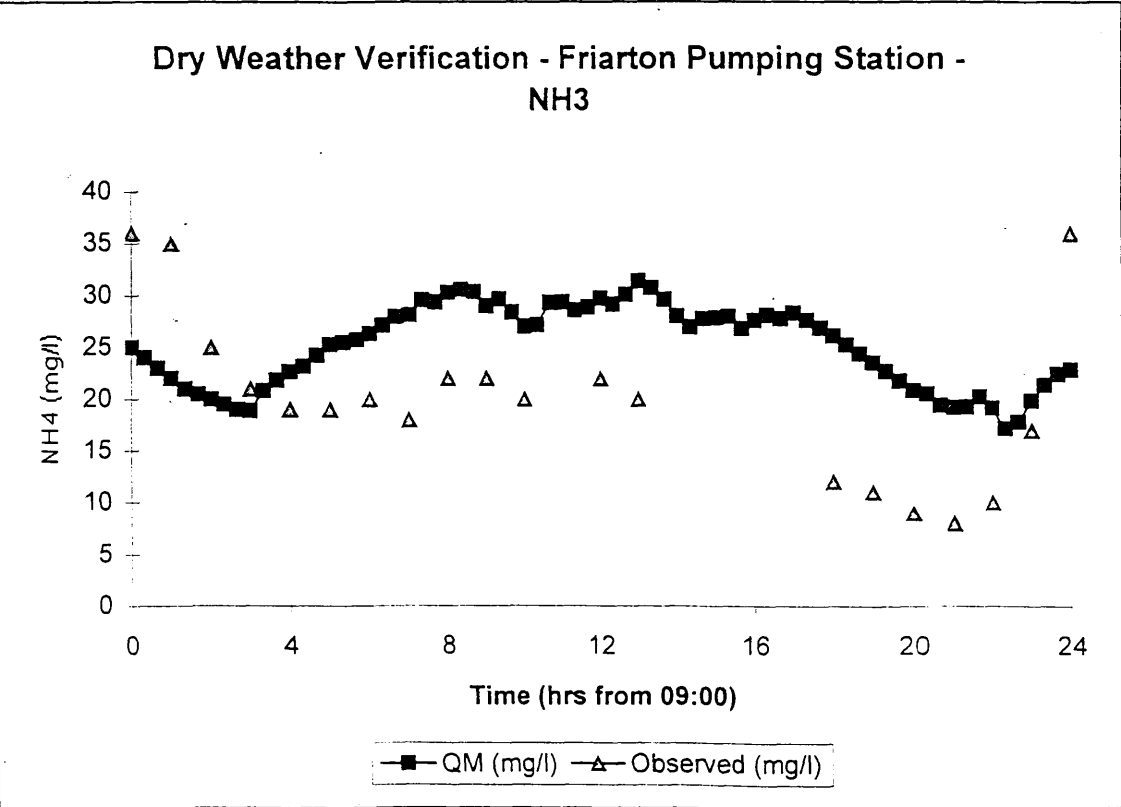


Fig. D.19 TSS at System Outfall (-QM)

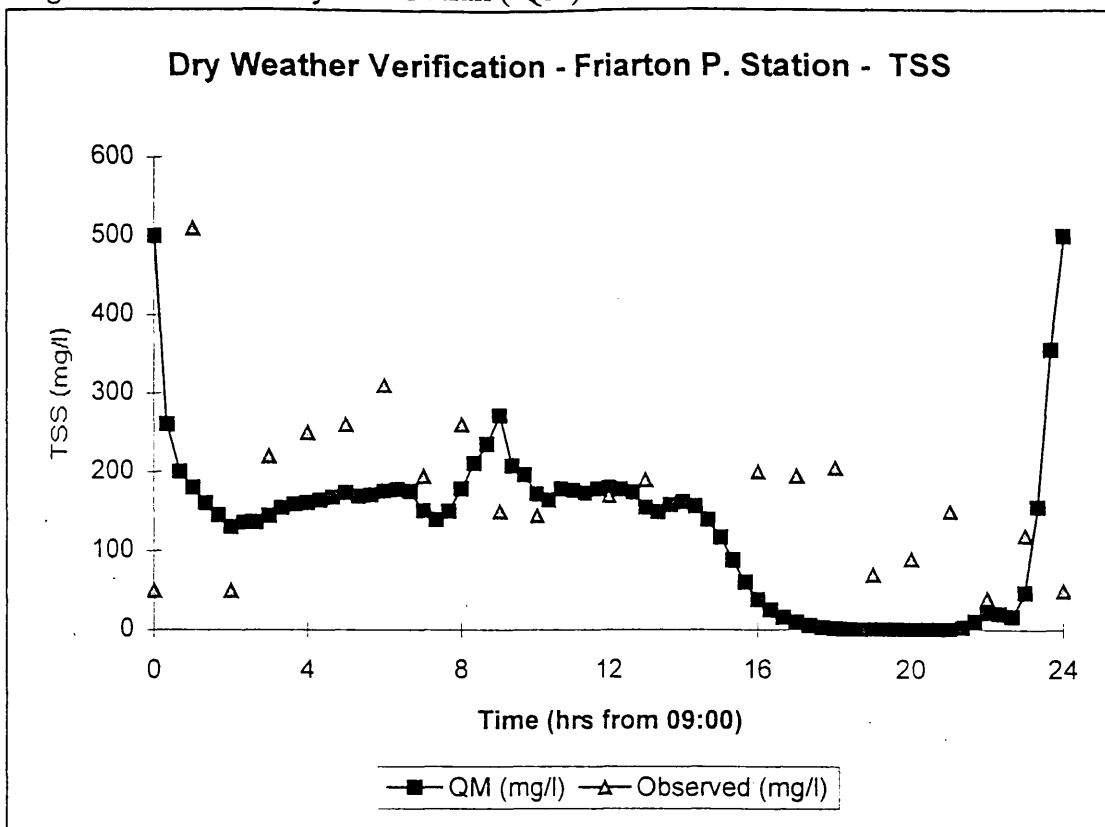


Fig. D.20 COD at System Outfall (-QM)

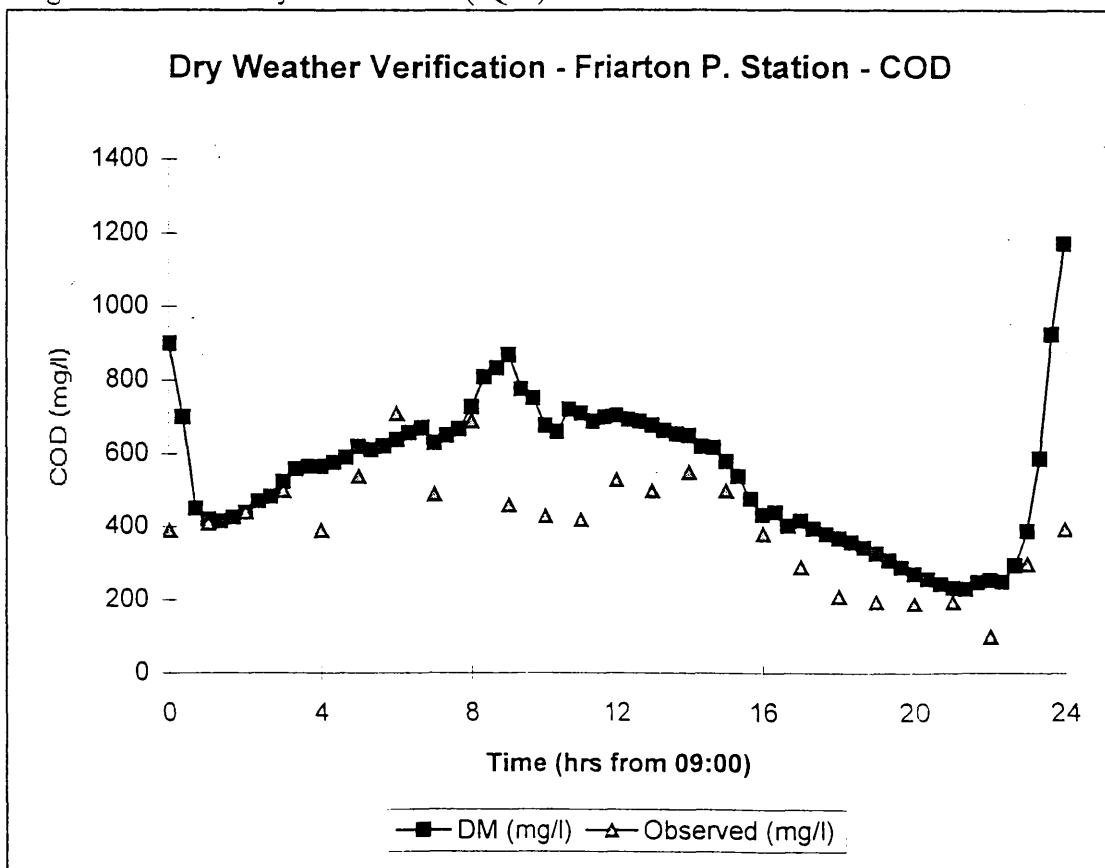


Fig.D.21 Modelled and Observed BOD – North Muirton (DWF)

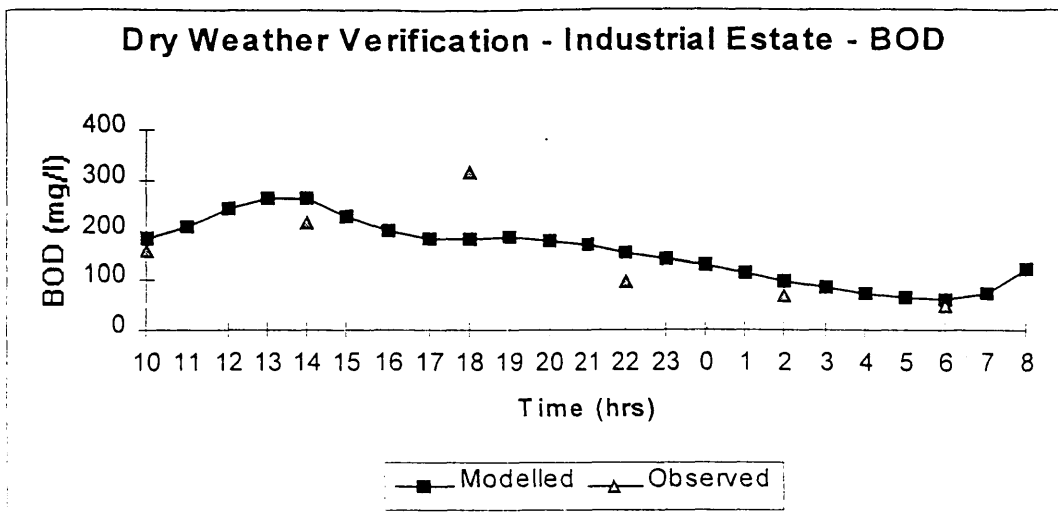


Fig. D.22 Modelled and Observed BOD – Bridgend (DWF)

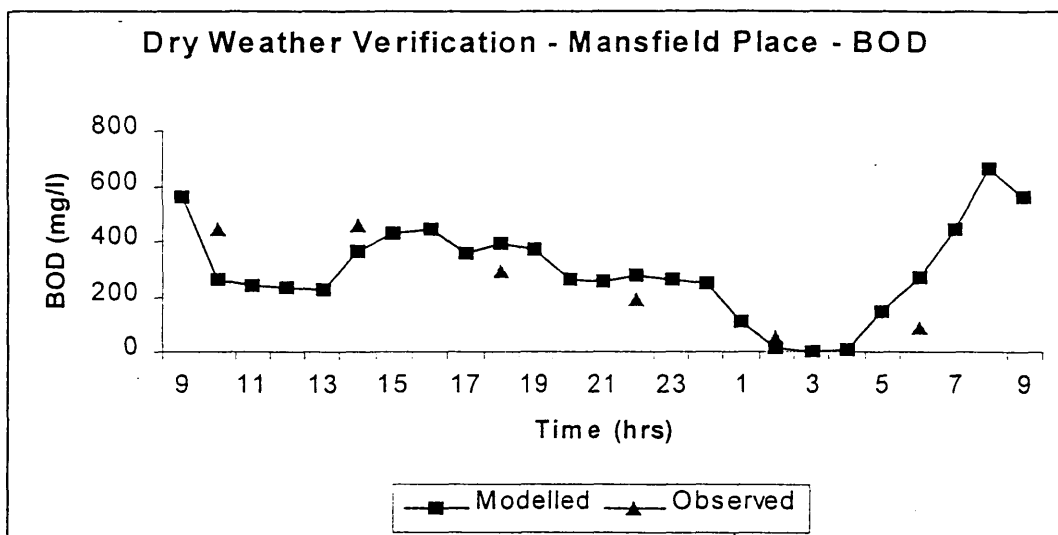


Fig. D.23 Modelled and Observed BOD – Tullton (DWF)

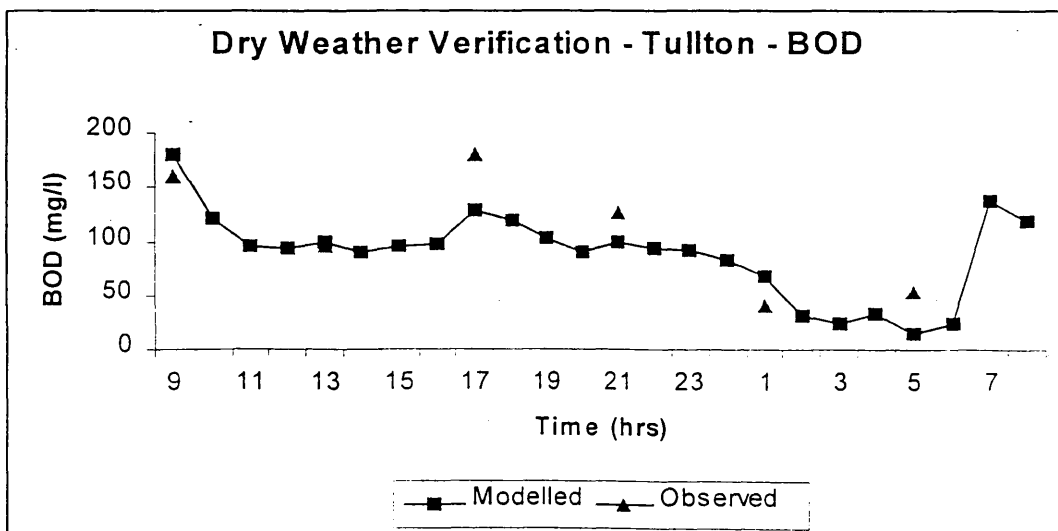


Fig.D.24 Modelled and Observed BOD – System Outfall (DWF)

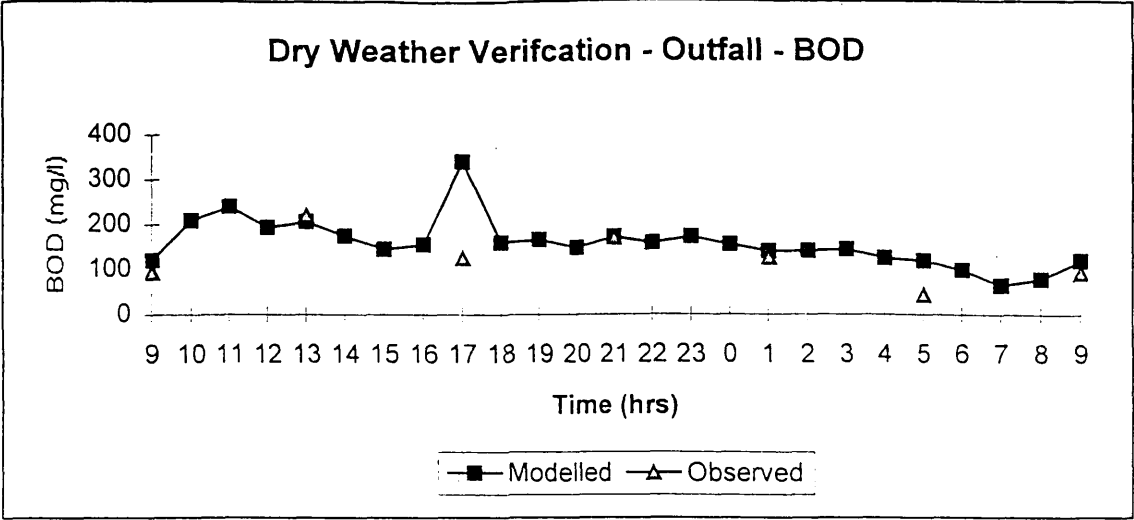


Fig. D.25 Modelled and Observed Flow – Bridgend (24/8/95)

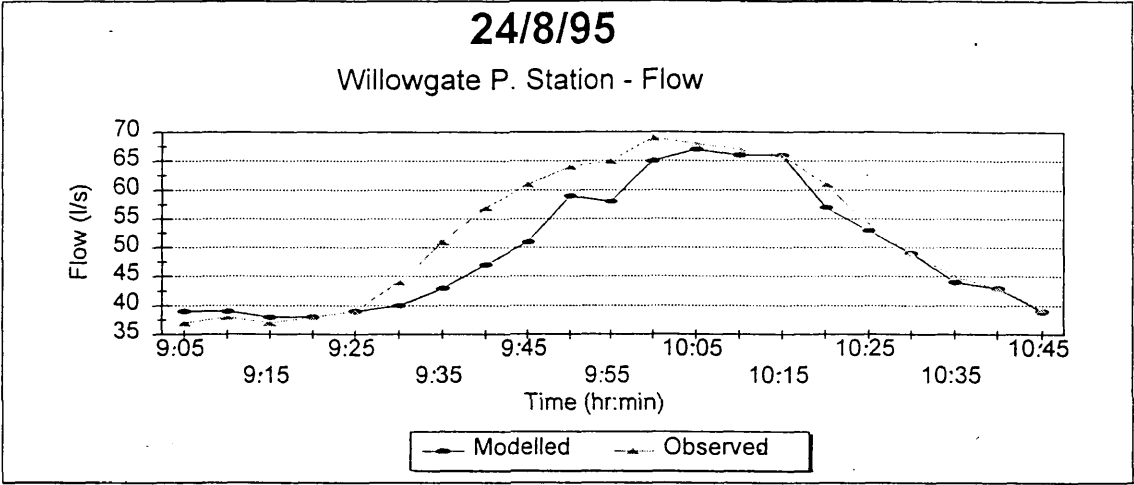


Fig. D.26 Modelled and Observed BOD - Bridgend (24/8/95)

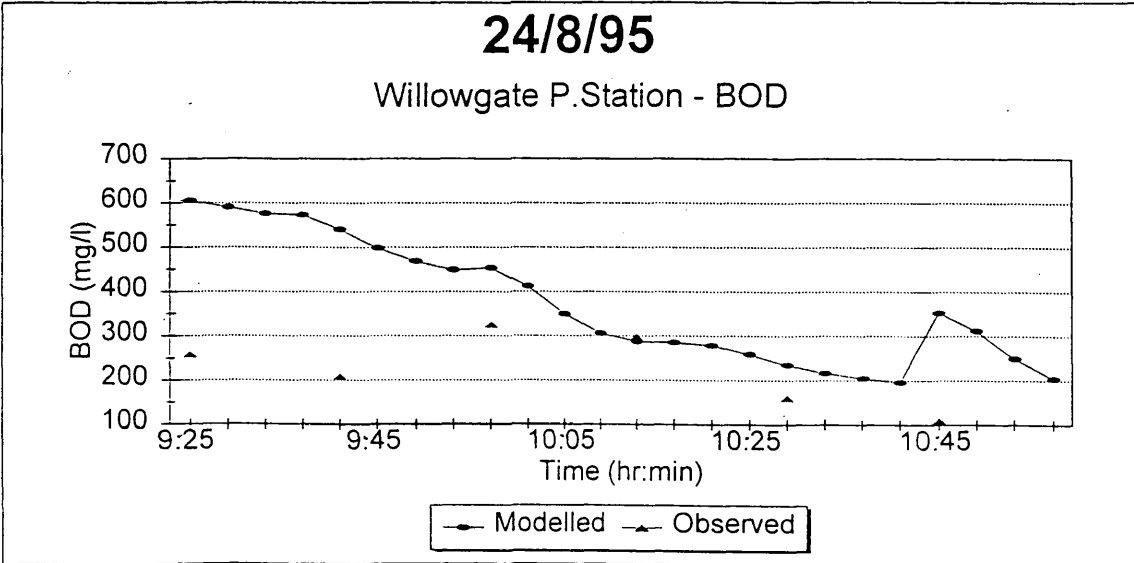


Fig. D.27 Modelled and Observed Flow – South Inch Outfall (24/8/95)

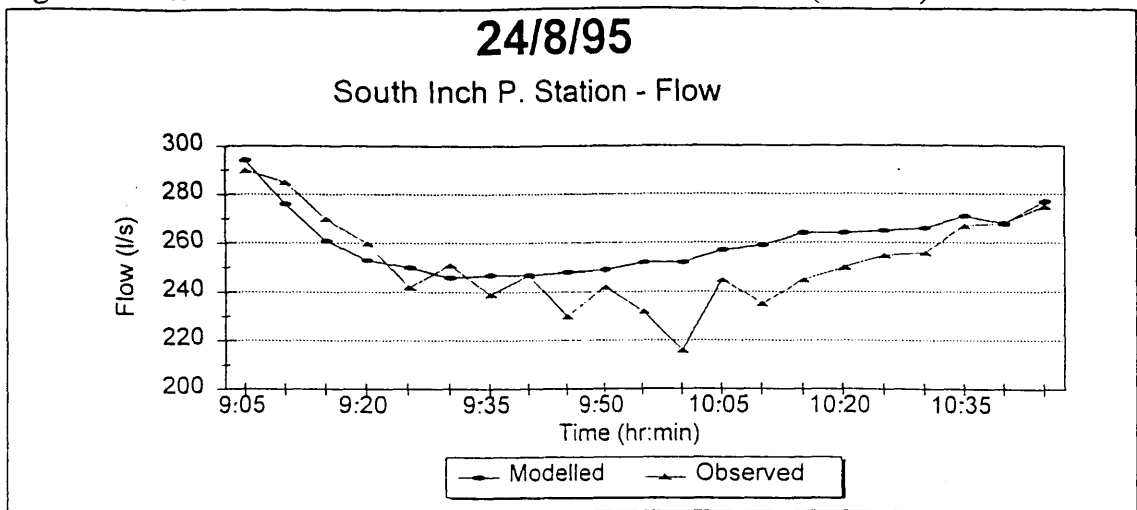


Fig. D.28 Modelled and Observed BOD – South Inch Outfall (24/8/95)

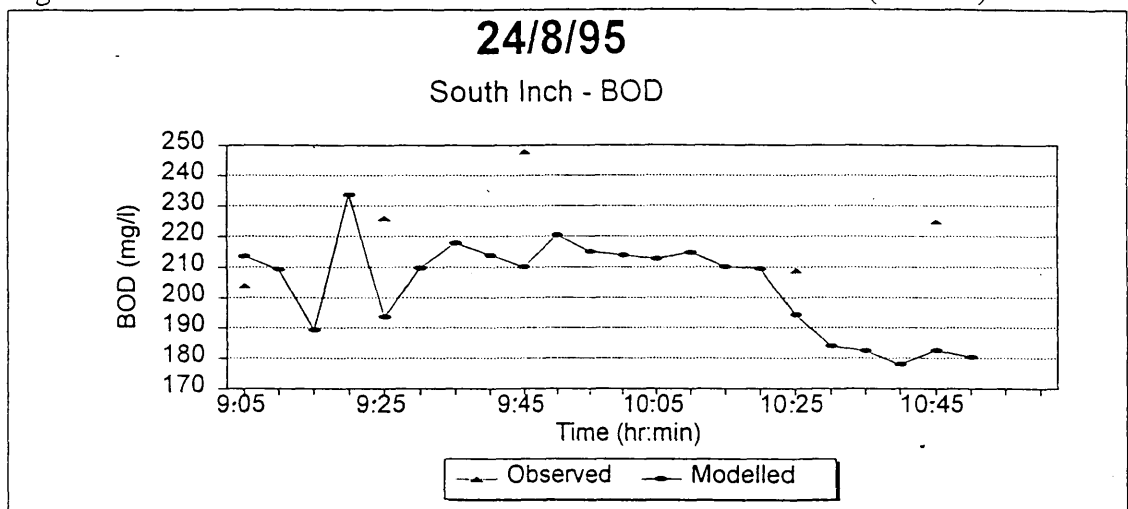


Fig. D.29 Modelled and Observed Flow – South Inch Outfall (29/8/95)

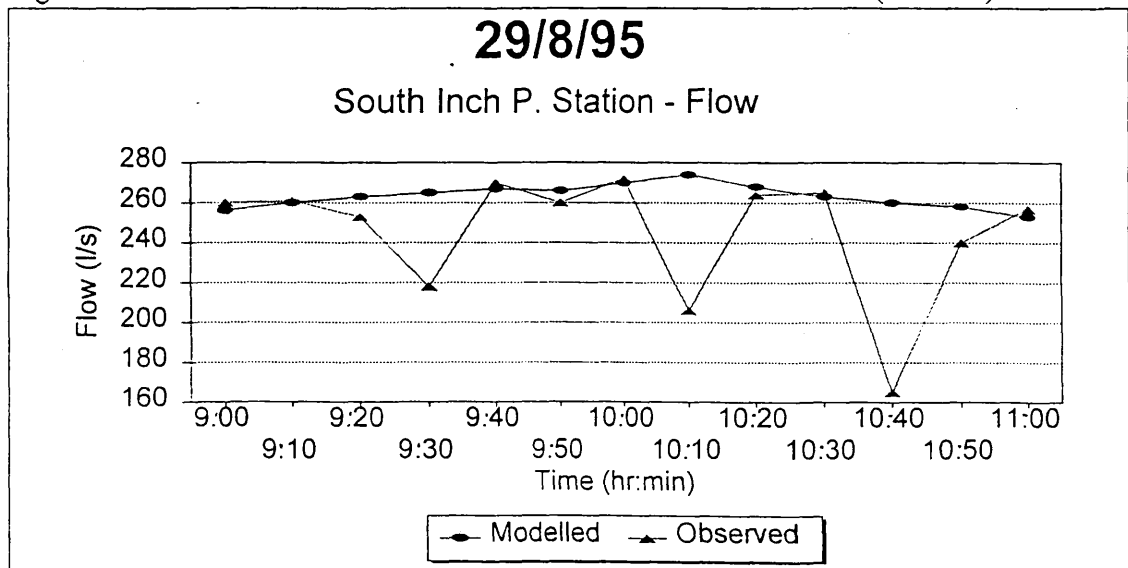


Fig. D.30 Modelled and Observed BOD – South Inch Outfall (29/8/95)

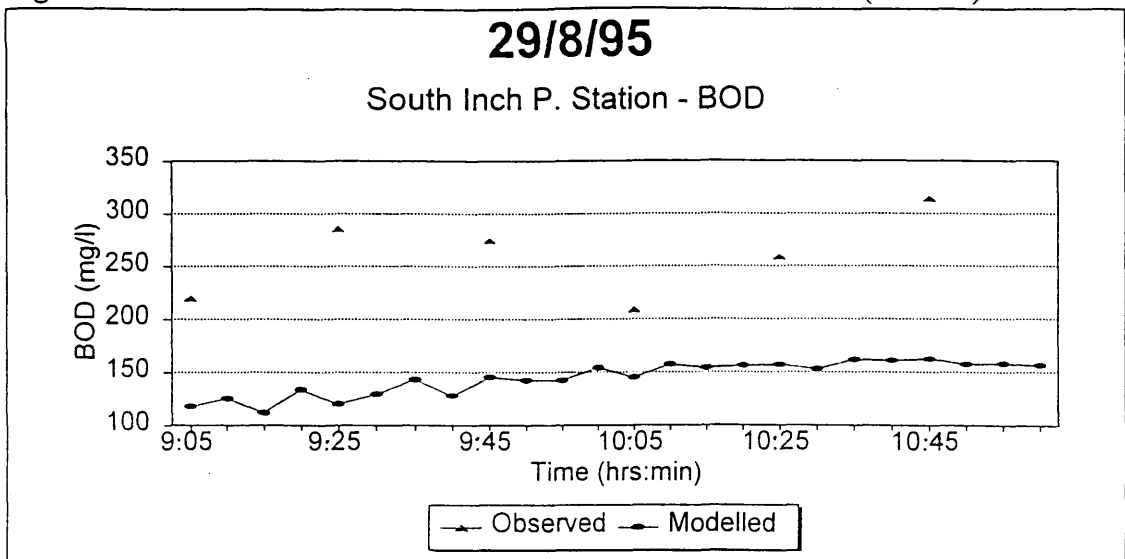


Fig. D.31 Modelled and Observed Flow – Bridgend (31/5/95)

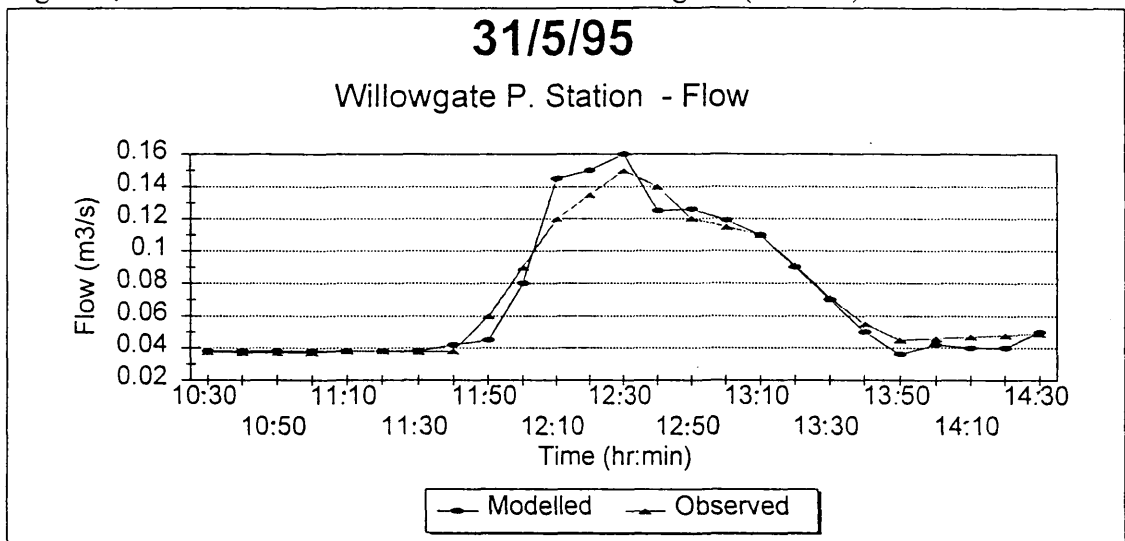


Fig. D.32 Modelled and Observed BOD – Bridgend (31/5/95)

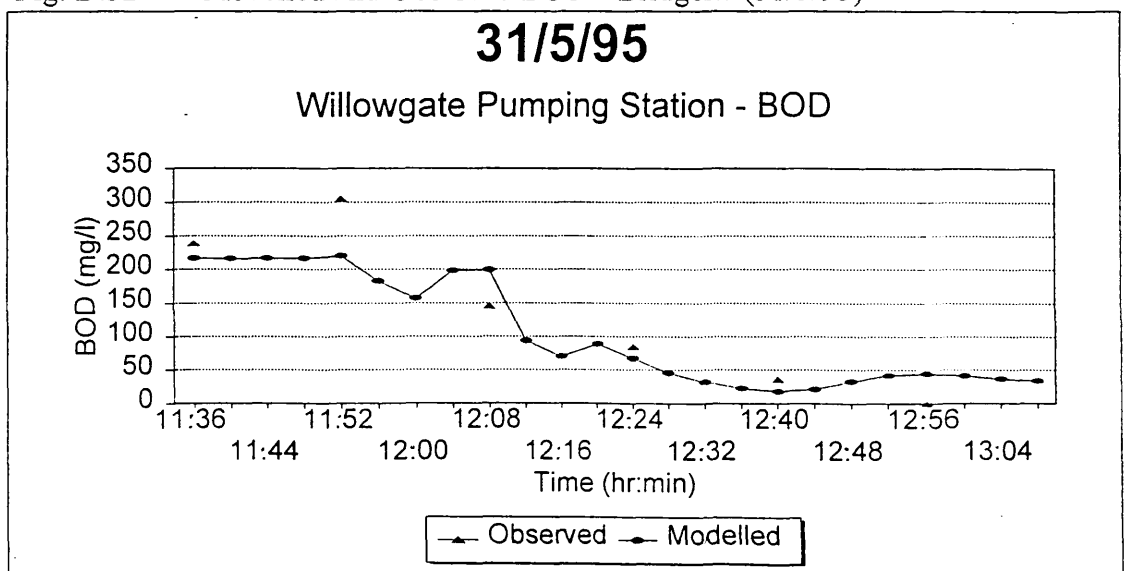


Fig.D.33 Modelled and Observed Flow – Craigie (31/5/95)

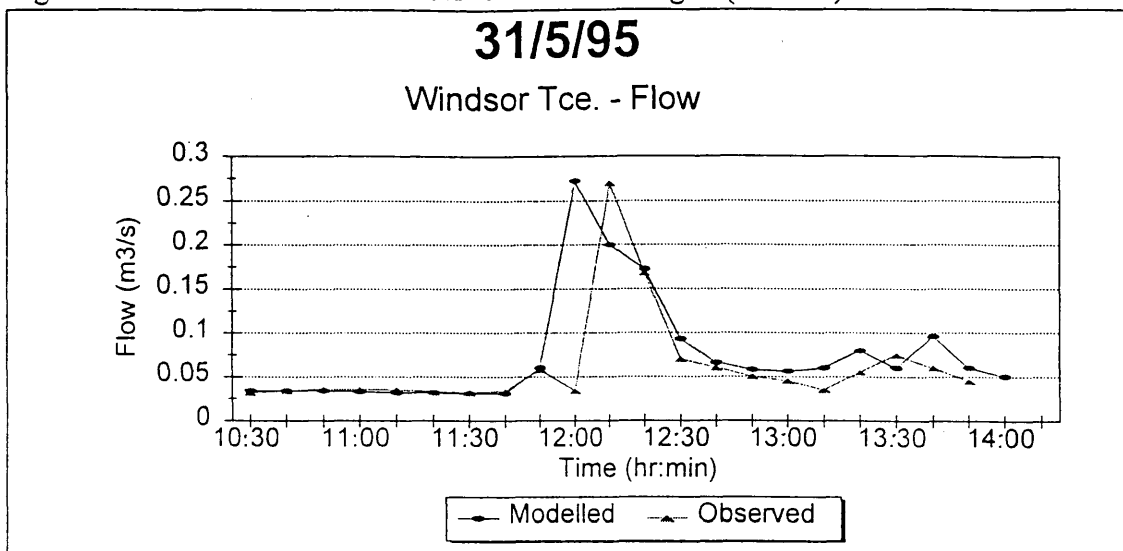


Fig.D.34 Modelled and Observed BOD – Craigie (31/5/95)

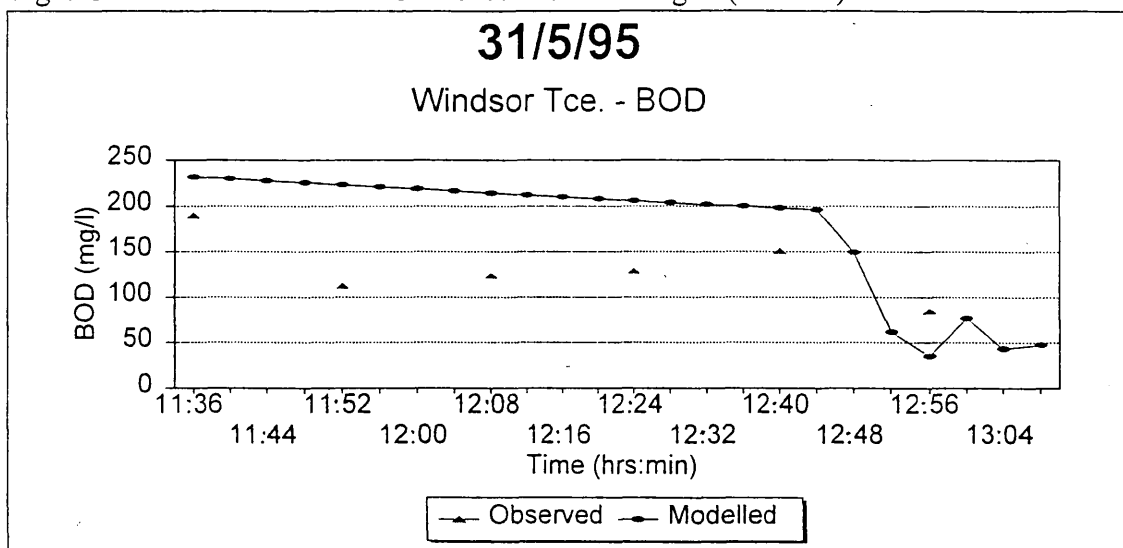


Fig.D.35 Modelled and Observed BOD – South Inch Outfall (31/5/95)

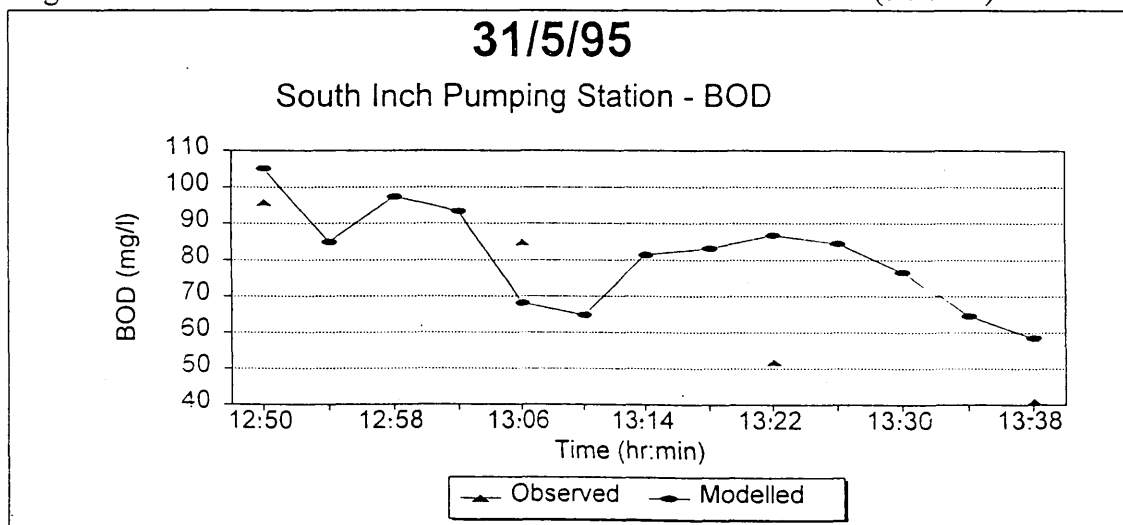


Fig.D.36 Initialisation Period Sensitivity Testing

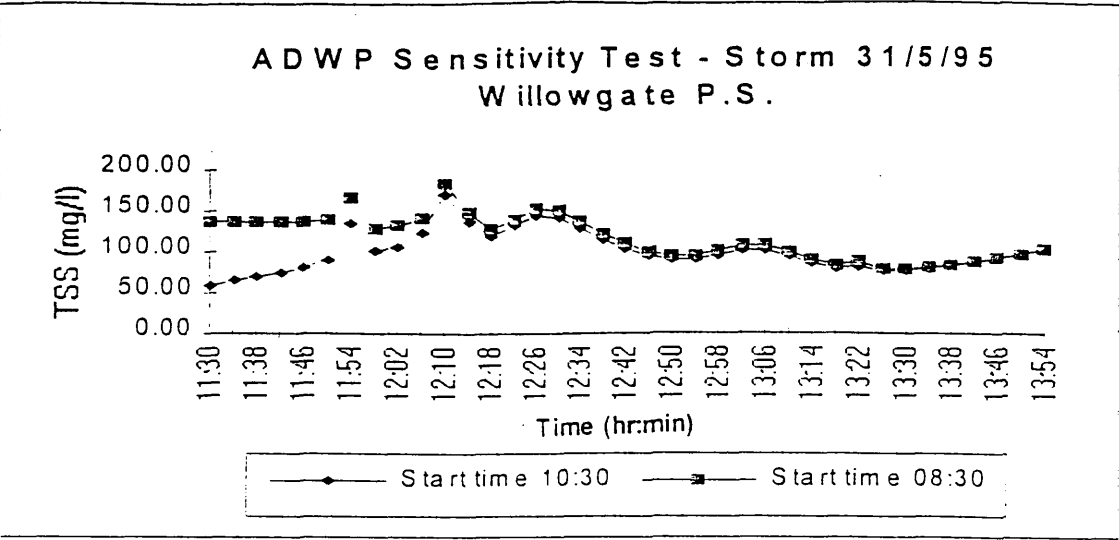


Fig. D.37 ADWP Sensitivity Test

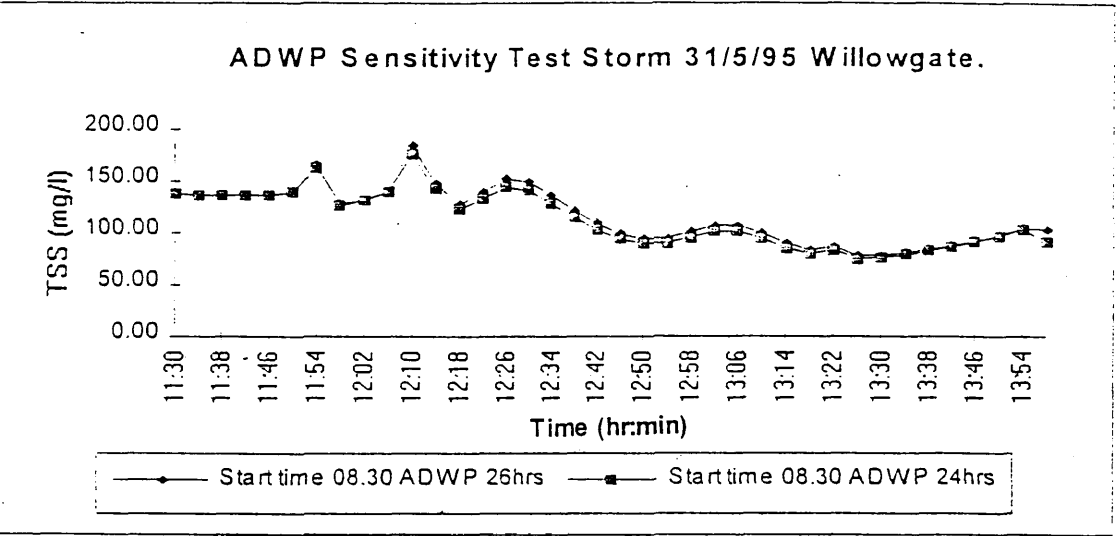
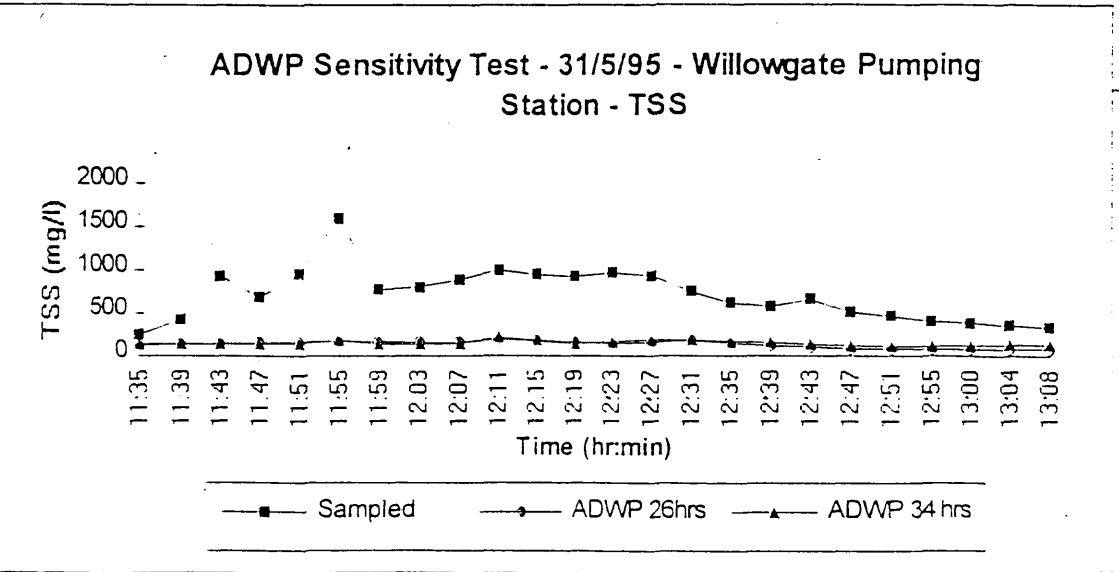


Fig. D.38 Maximum Possible ADWP Analysis (TSS)



D.39 Maximum Possible ADWP Analysis (COD)

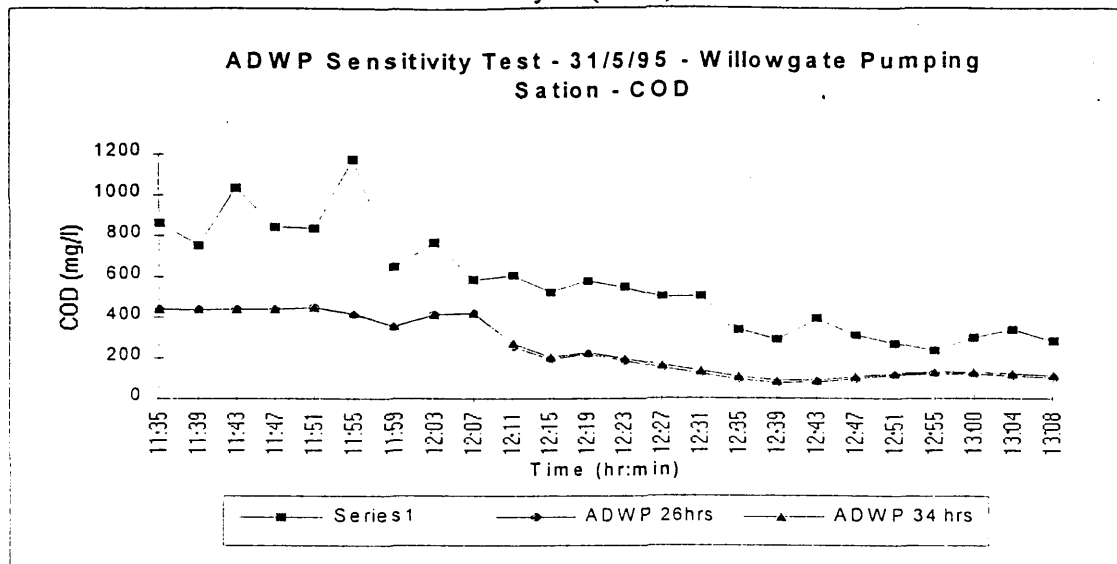


Fig. D.40 "Infiltration" and "No Infiltration" Model Comparison – Flow

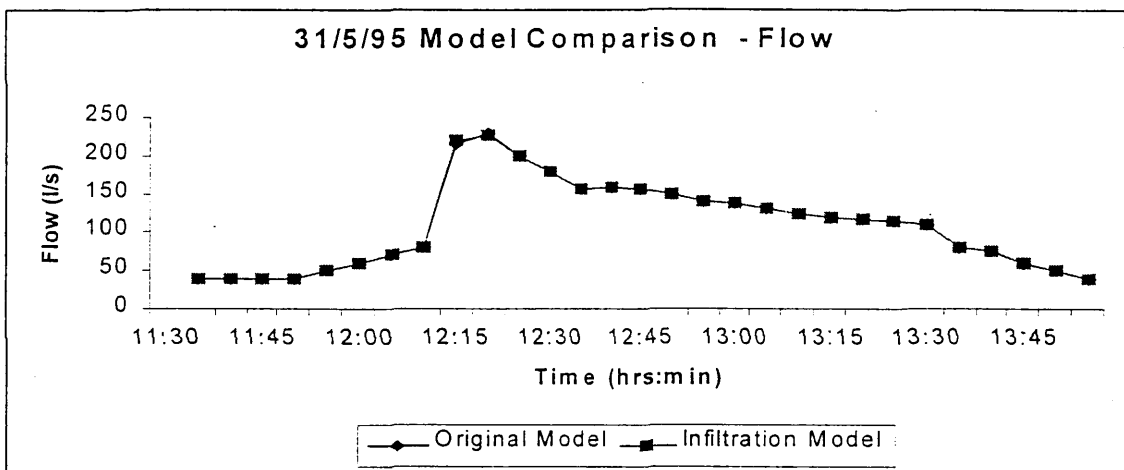


Fig. D.41 "Infiltration" and "No Infiltration" Model Comparison – TSS

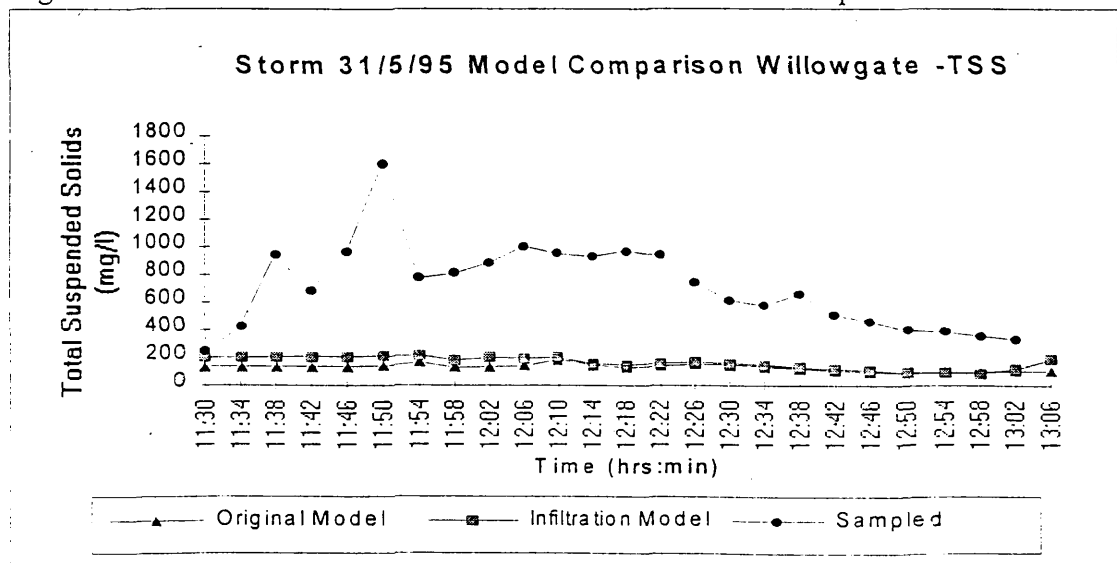


Fig. D.42 "Infiltration" and "No Infiltration" Model Comparison – COD

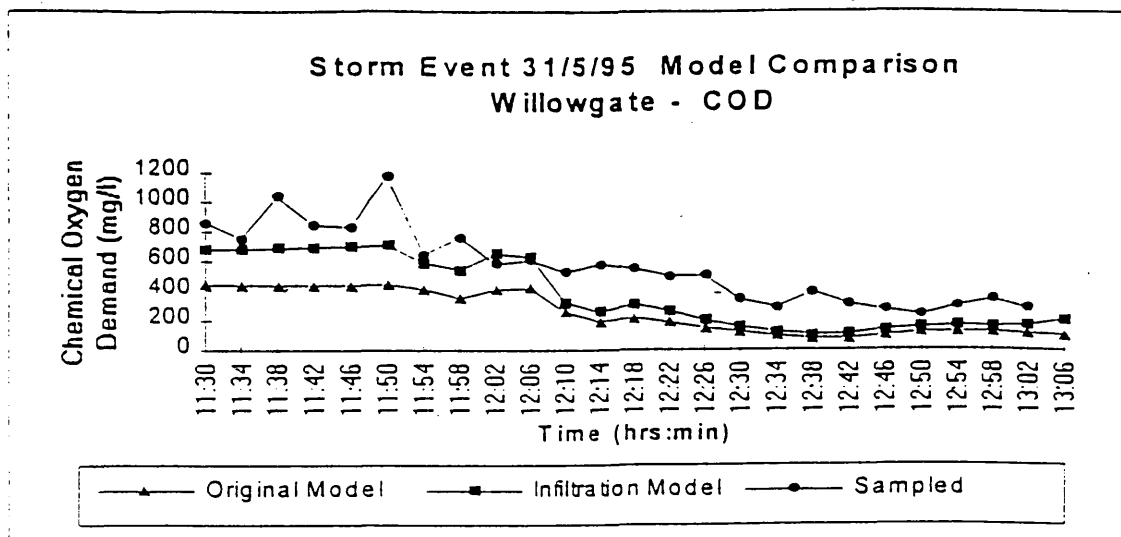


Fig.D.43 Diameter Sensitivity Analysis - TSS (Bridgend - DWF)

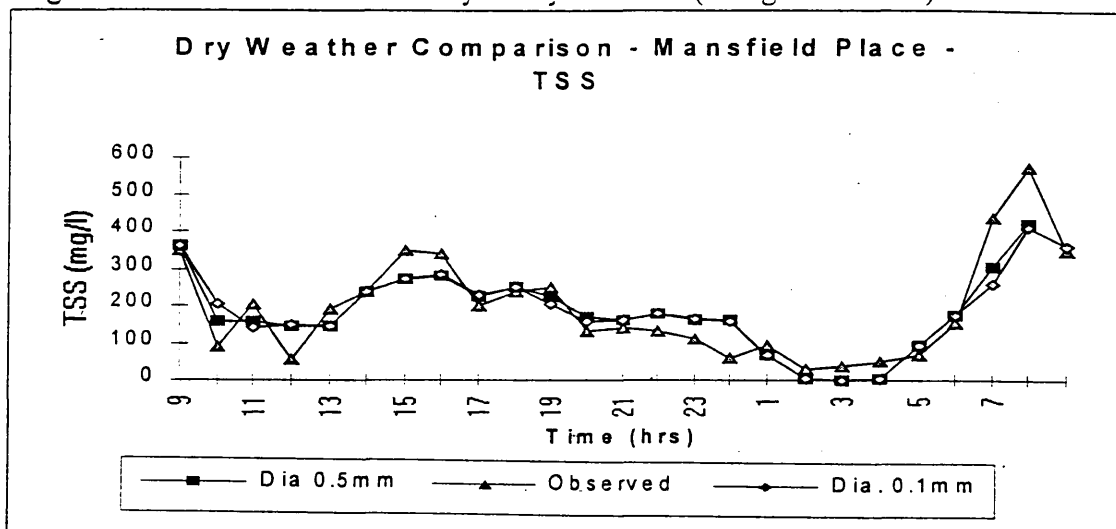


Fig.D.44 Diameter Sensitivity Analysis - COD (Bridgend - DWF)

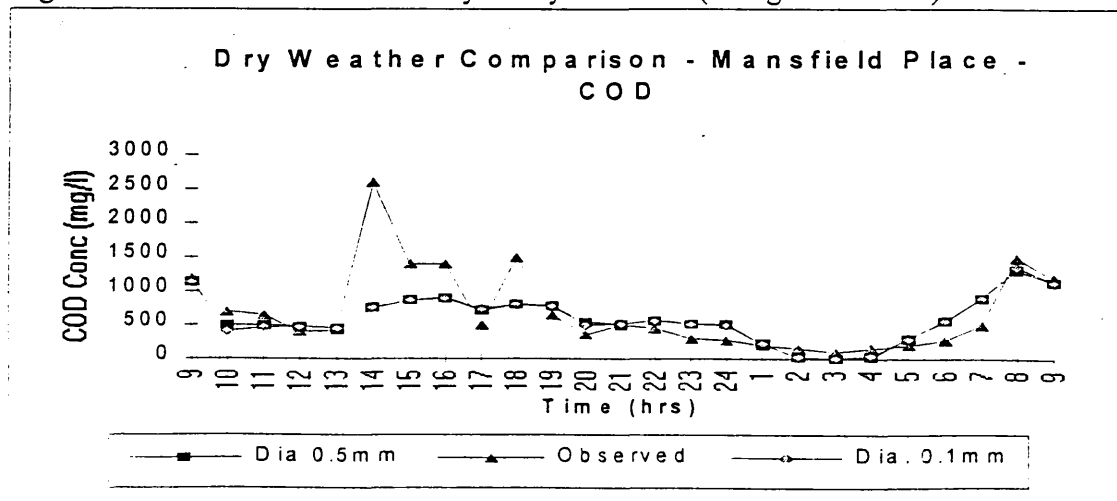


Fig. D.45 Diameter Sensitivity Analysis - TSS (Outfall - DWF)

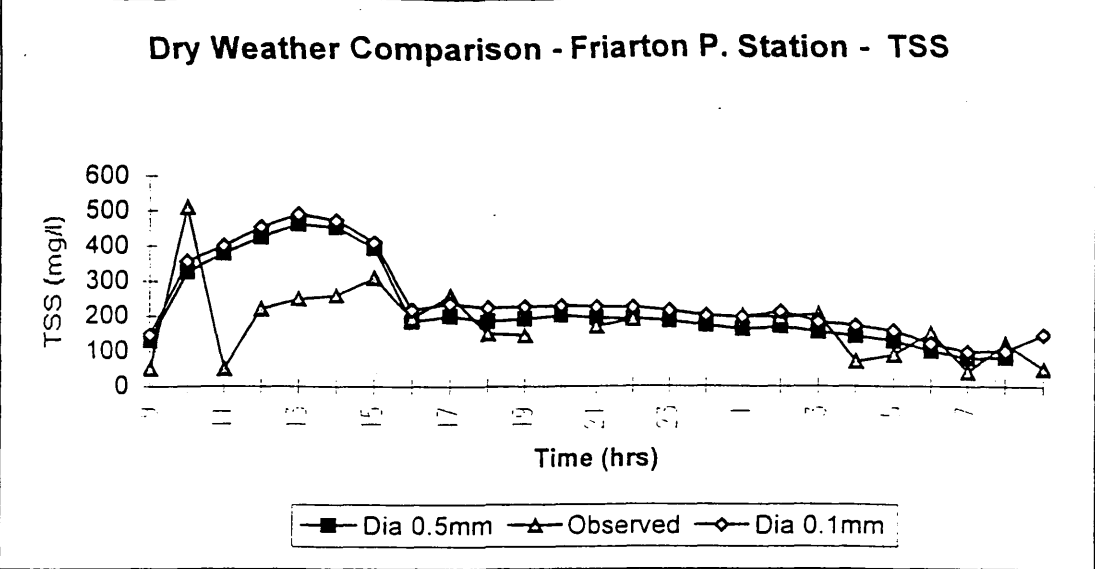


Fig.D.46 Diameter Sensitivity Analysis - COD (Outfall - DWF)

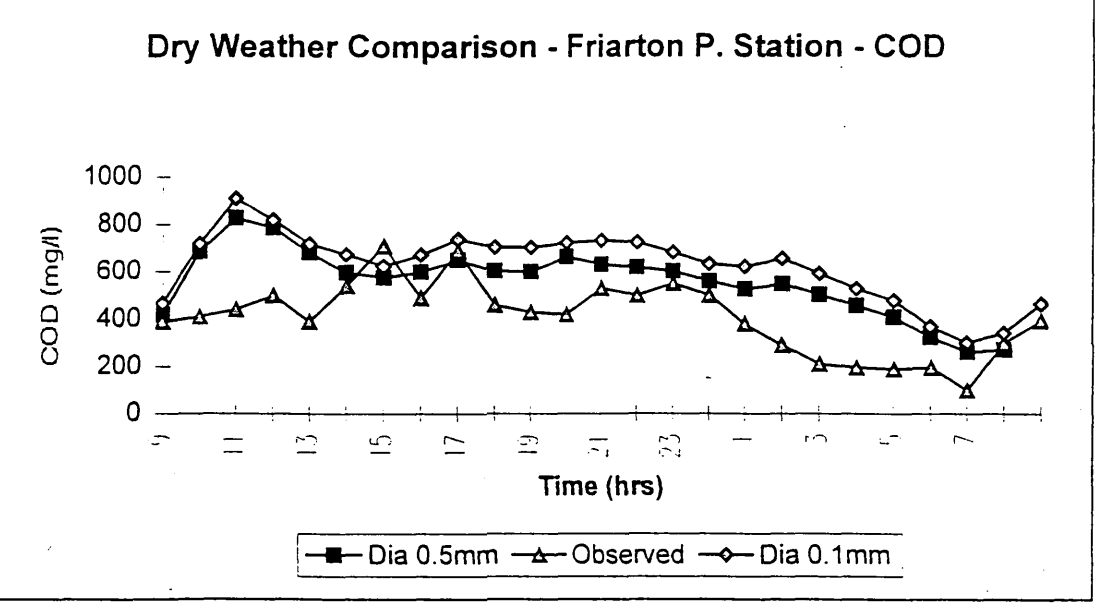


Fig. D.47 Diameter Analysis - TSS (Storm)

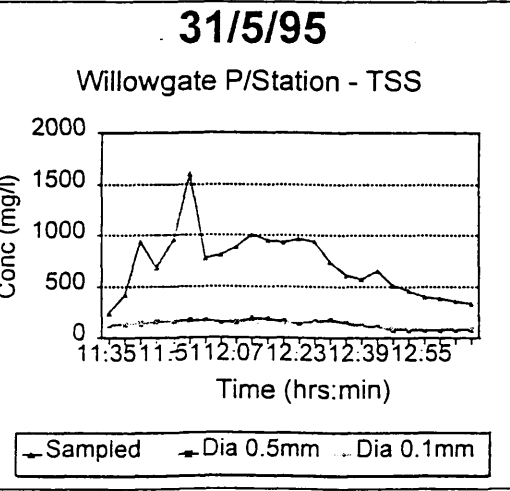


Fig. D.48 Diameter Analysis - COD Storm

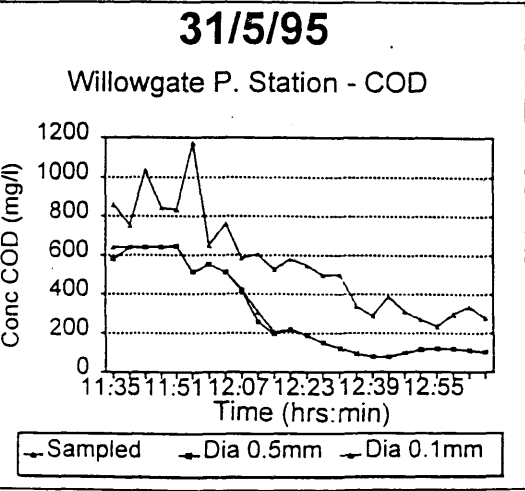


Fig. D.49 Specific Gravity Sensitivity Analysis - TSS (Bridgend - DWF)

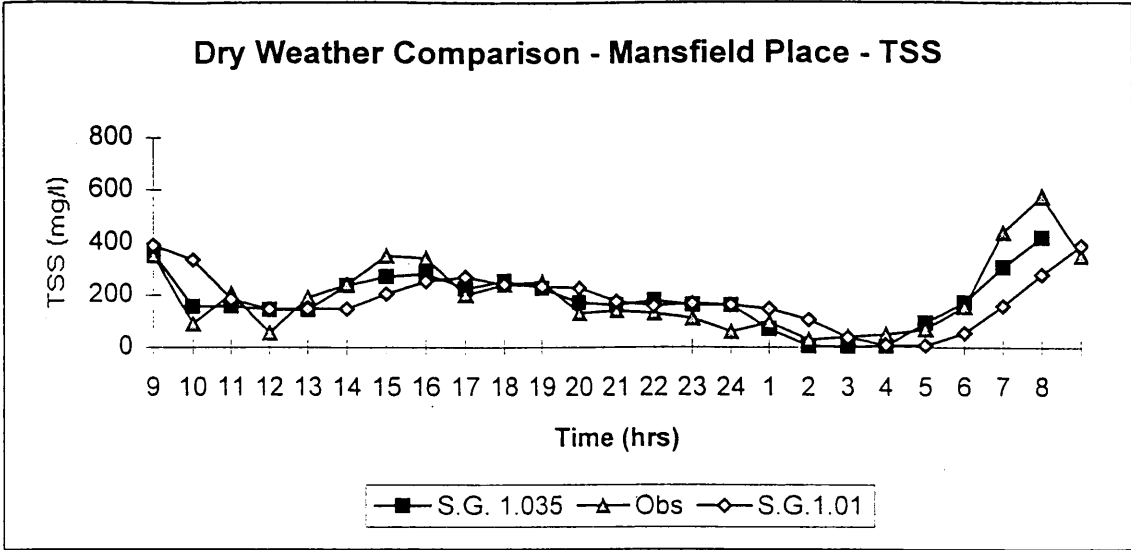


Fig. D.50 Specific Gravity Sensitivity Analysis - COD (Bridgend - DWF)

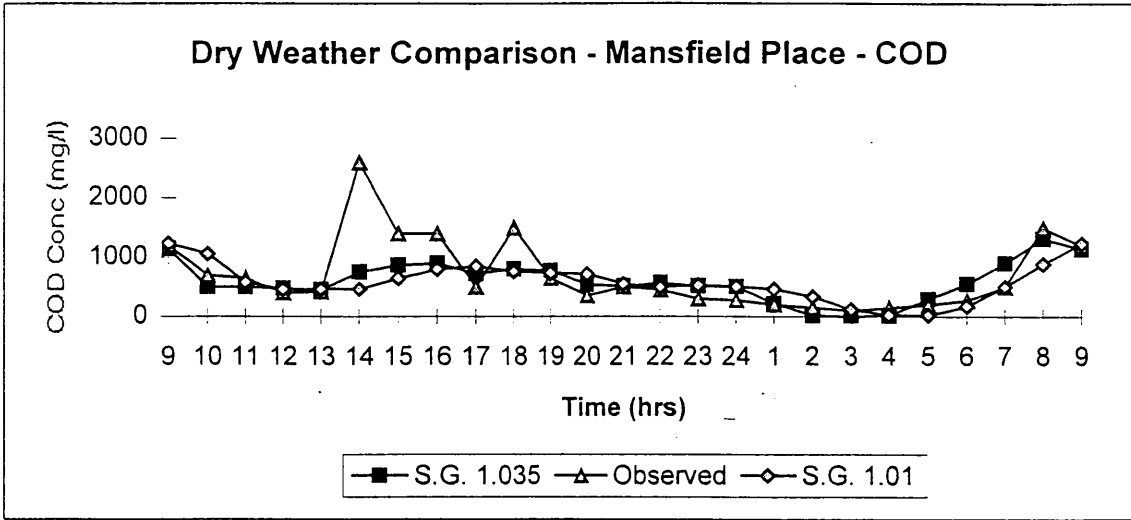


Fig. D.51 Specific Gravity Sensitivity Analysis - TSS (Outfall - DWF)

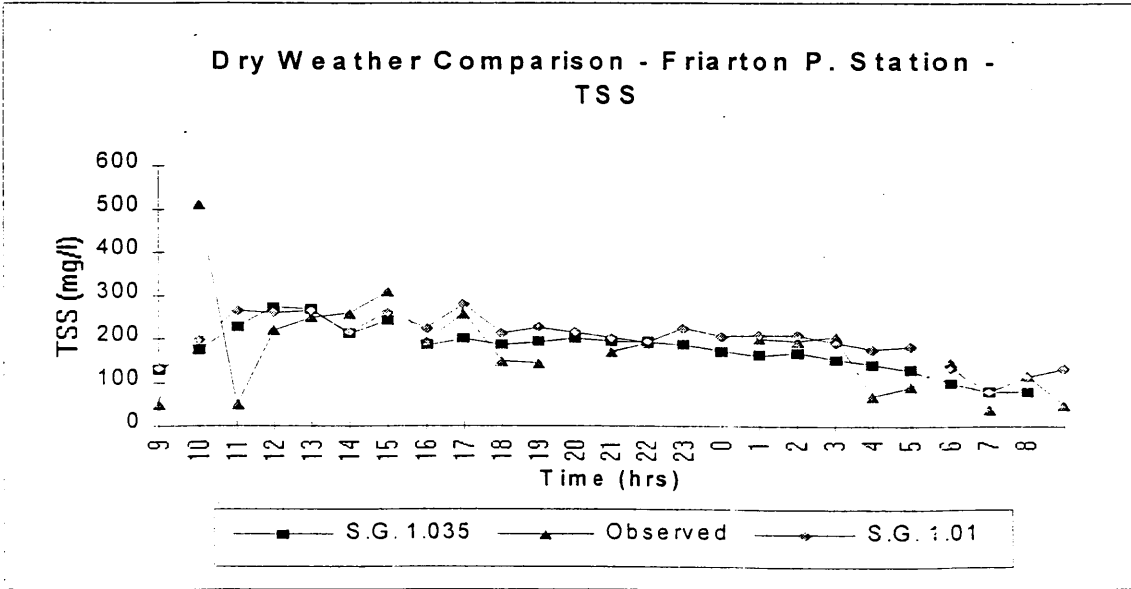


Fig. D.52 Specific Gravity Sensitivity Analysis - COD (Outfall - DWF)

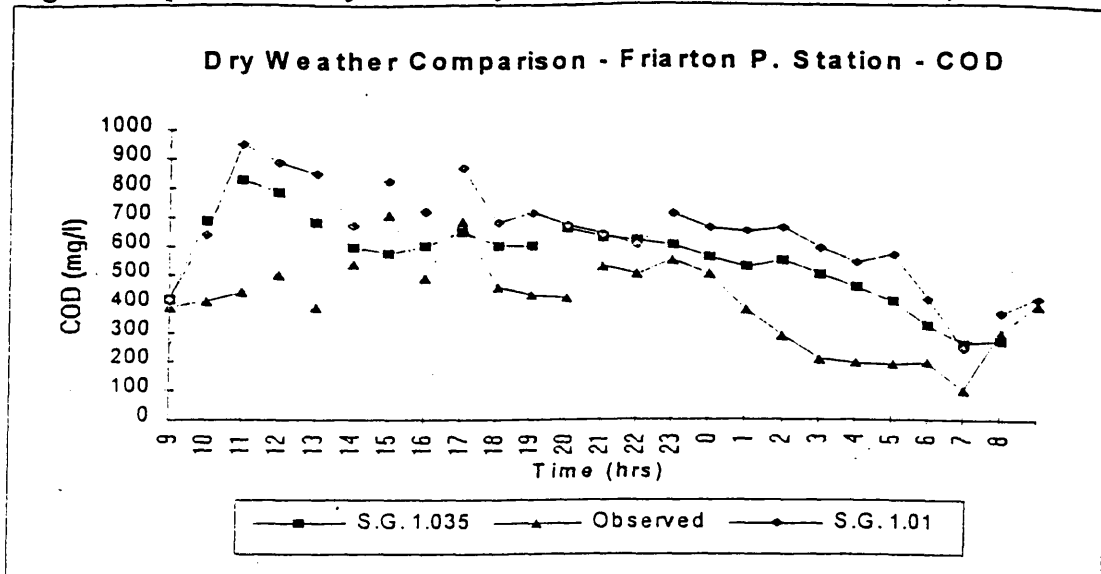


Fig. D.53 S.G. Analysis - TSS (Storm) Fig. D.54 S.G. Analysis - COD (Storm)

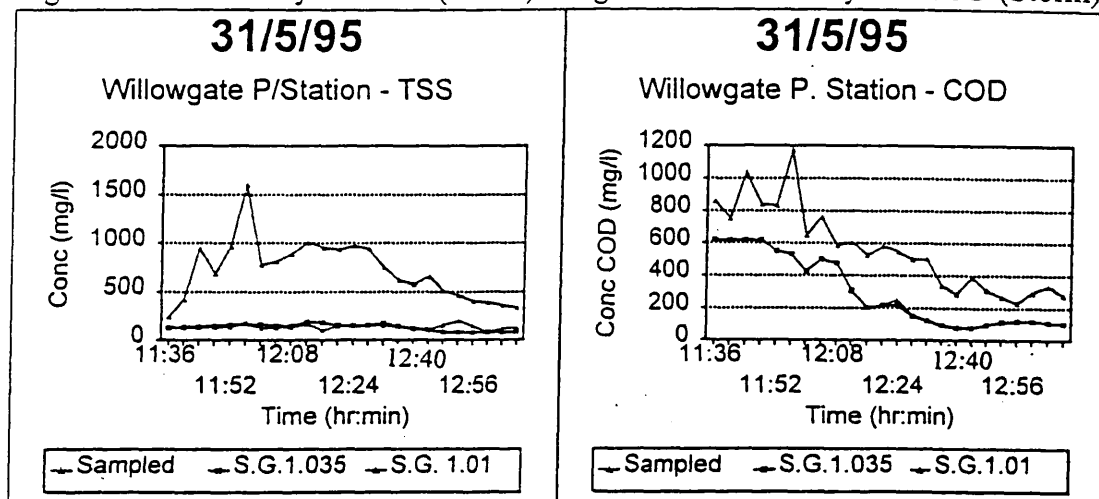


Fig. D.55 Additional S.G Analysis - TSS (Outfall - DWF)

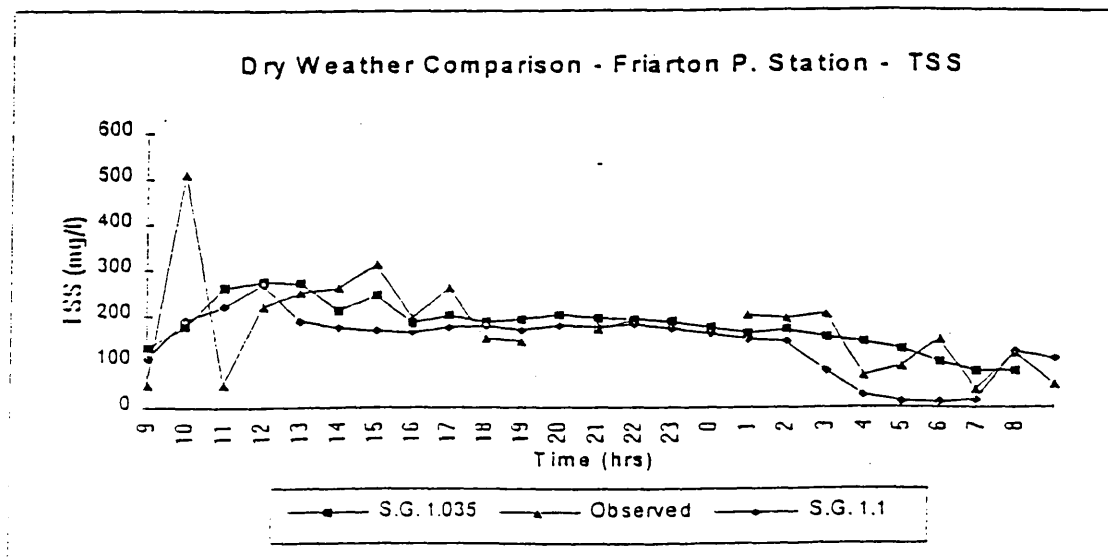


Fig. D.56 Additional S.G. Analysis – COD (Outfall – DWF)

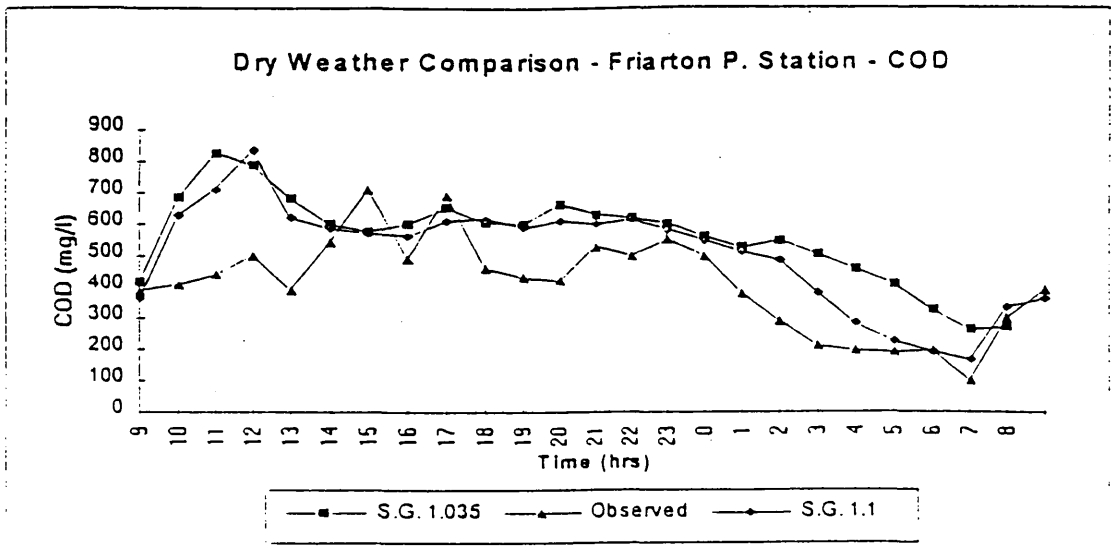


Fig. D.57 Additional S.G. Analysis - TSS (Storm)

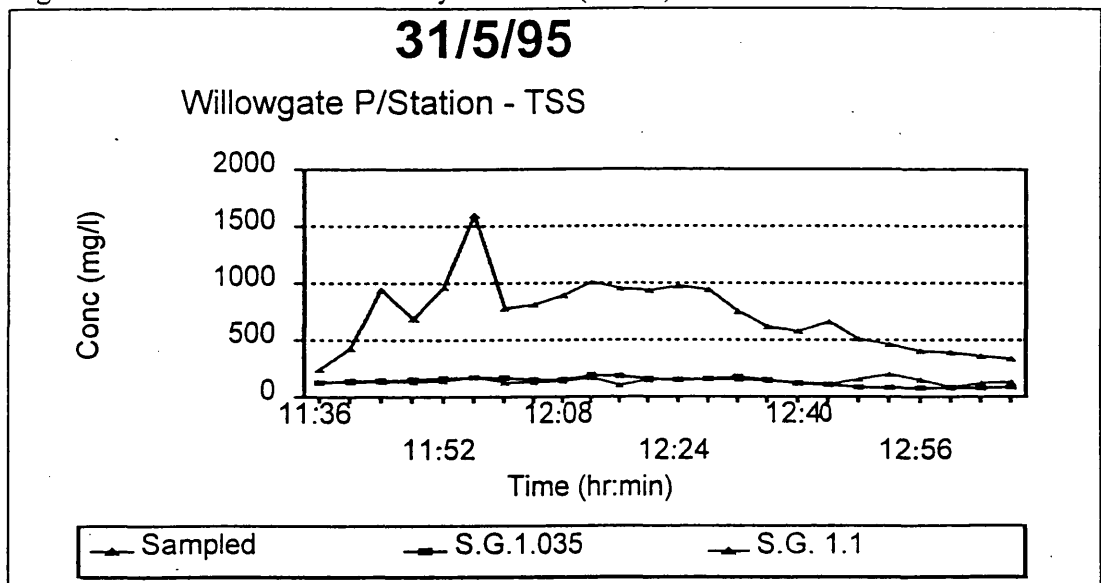
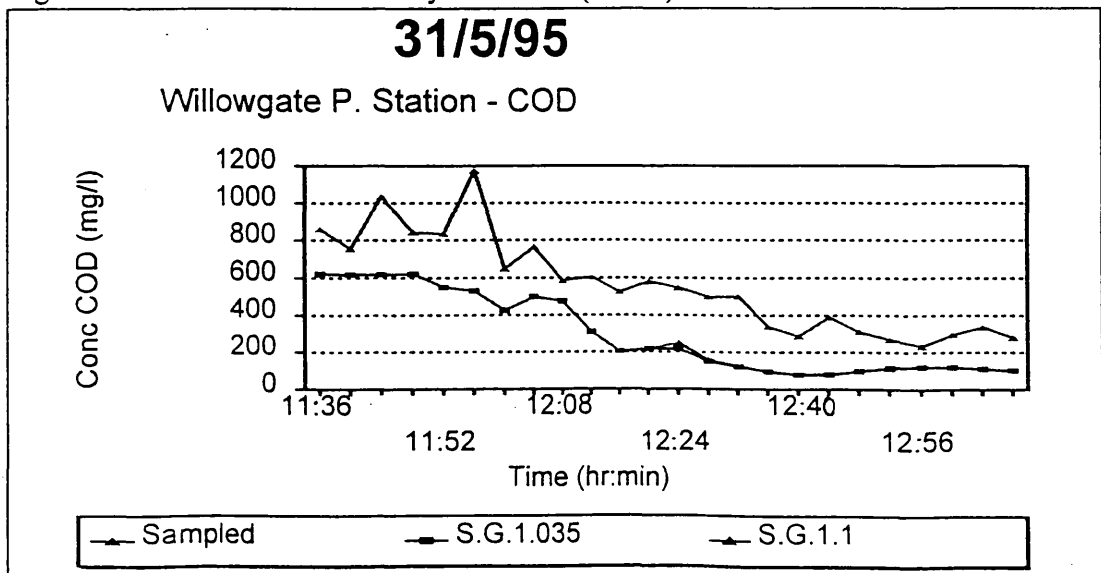


Fig. D.58 Additional S.G. Analysis - COD (Storm)



Appendix E

Waste Water Treatment Plant Modelling - Supporting Graphs

List of Figures

E.1	TSS in Primary Tank Effluent (STOAT - DWF)	E5
E.2	BOD in Primary Tank Effluent (STOAT - DWF)	E5
E.3	Ammonia in Primary Tank Effluent (STOAT - DWF)	E5
E.4	TSS in Activated Sludge Effluent (STOAT - DWF)	E6
E.5	BOD in Activated Sludge Effluent (STOAT - DWF)	E6
E.6	Ammonia in Activated Sludge Effluent (STOAT - DWF)	E6
E.7	TSS in Primary Tank Effluent (STOAT - Storm)	E7
E.8	BOD in Primary Tank Effluent (STOAT - Storm)	E7
E.9	Ammonia in Primary Tank Effluent (STOAT - Storm)	E7
E.10	TSS in Primary Tank Effluent (GPS-X - DWF)	E8
E.11	BOD in Primary Tank Effluent (GPS-X - DWF)	E8
E.12	Ammonia in Primary Tank Effluent (GPS-X - DWF)	E8
E.13	TSS in Activated Sludge Effluent (GPS-X - DWF)	E9
E.14	BOD in Activated Sludge Effluent (GPS-X - DWF)	E9
E.15	Ammonia in Activated Sludge Effluent (GPS-X - DWF)	E9
E.16	TSS in Primary Tank Effluent (GPS-X - Storm)	E10
E.17	BOD in Primary Tank Effluent (GPS-X - Storm)	E10
E.18	Ammonia in Primary Tank Effluent (GPS-X - Storm)	E10
E.19	TSS in Primary Tank Effluent (STOAT & GPS-X - Storm)	E11
E.20	BOD in Primary Tank Effluent (STOAT & GPS-X - Storm)	E11
E.21	Storm Tank Calibrations - Event One - Flow (26/10/95)	E11
E.22	Storm Tank Calibrations - Event One - TSS (26/10/95)	E12
E.23	Storm Tank Calibrations - Event One - BOD (26/10/95)	E12
E.24	Storm Tank Calibrations - Event One - Ammonia (26/10/95)	E12
E.25	Storm Tank Calibrations - Event Two - Flow (31/10/95)	E13
E.26	Storm Tank Calibrations - Event Two - TSS (31/10/95)	E13
E.27	Storm Tank Calibrations - Event Two - BOD (31/10/95)	E13
E.28	Storm Tank Calibrations - Event Two - Ammonia (31/10/95)	E14
E.29	WTP Disruptive Period Analysis - Flow	E14

List of Figures

E.30	WTP Disruptive Period Analysis - TSS and BOD	E14
E.31	Test 1c (Influent of 600l/s for 8 days) – WTP Effluent - Flow	E15
E.32	Test 1c (Influent of 600l/s for 8 days) – WTP Effluent – TSS & BOD	E15
E.33	Test 1c (Influent of 600l/s for 8 days) – Heterotrophic Biomass in Activated Sludge Tank	E15
E.34	Test 2c (Influent of 800l/s for 8 days) – WTP Effluent – Flow	E16
E.35	Test 2c (Influent of 800l/s for 8 days) – WTP Effluent – TSS & BOD	E16
E.36	Test 2c (Influent of 800l/s for 8 days) – Heterotrophic Biomass in Activated Sludge Tank	E16
E.37	Flow from Secondary Settlement Tank - Sludge Loss Analysis	E17
E.38	TSS & BOD in from Secondary Settlement Tank - Sludge Loss Analysis	E17
E.39	Test 3a (Influent of 1000l/s for 4 days) – WTP Effluent - Flow	E17
E.40	Test 3a (Influent of 1000l/s for 4 days) – WTP Effluent – TSS & BOD	E18
E.41	Test 3a (Influent of 1000l/s for 4 days) Heterotrophic Biomass in Activated Sludge Tank	E18
E.42	Test 3b (Influent of 1000l/s for 6 days) – WTP Effluent - Flow	E18
E.43	Test 3b (Influent of 1000l/s for 6 days) – WTP Effluent – TSS & BOD	E19
E.44	Test 3b (Influent of 1000l/s for 6 days) – Heterotrophic Biomass in Activated Sludge Tank	E19
E.45	Test 3c (Influent of 1000l/s for 8 days) – WTP Effluent - Flow	E19
E.46	Test 3c (Influent of 1000l/s for 8 days) – WTP Effluent – TSS & BOD	E20

List of Figures

E.47	Test 3c (Influent of 1000l/s for 8 days) Heterotrophic Biomass in Activated Sludge Tank	E20
E.48	Influent of 1000l/s for 1 day - WTP Effluent - Flow	E20
E.49	Influent of 1000l/s for 1 day - WTP Effluent – BOD	E21
E.50	Influent of 1000l/s for 1 day - Heterotrophic Biomass in Activated Sludge Tank	E21
E.51	Influent of 1000l/s for 2 days - WTP Effluent - Flow	E21
E.52	Influent of 1000l/s for 2 days - WTP Effluent – BOD	E22
E.53	Influent of 1000l/s for 2 days - Heterotrophic Biomass in Activated Sludge Tank	E22
E.54	Influent of 1000l/s for 25 days - WTP Effluent - Flow	E22
E.55	Influent of 1000l/s for 25 days - WTP Effluent - BOD	E23
E.56	Influent of 1000l/s for 25 days - Heterotrophic Biomass in Activated Sludge Tank	E23
E.57	WTP Effluent (DWF) - Flow	E23
E.58	WTP Effluent (DWF) – Ammonia	E24
E.59	Influent of 600l/s for 4 days – WTP Effluent – Flow	E24
E.60	Influent of 600l/s for 4 days – WTP Effluent – Ammonia	E24
E.61	Influent of 600l/s for 4 days - Autotrophic Biomass in Activated Sludge Tank	E.25
E.62	Performance Comparison (600l/s and 700l/s for 4 days) – WTP Effluent - Ammonia	E25
E63	Performance Comparison (600l/s and 700l/s for 4 days) Autotrophic Biomass in Activated Sludge Tank	E25

Fig. E.1 TSS in Primary Tank Effluent (STOAT - DWF)

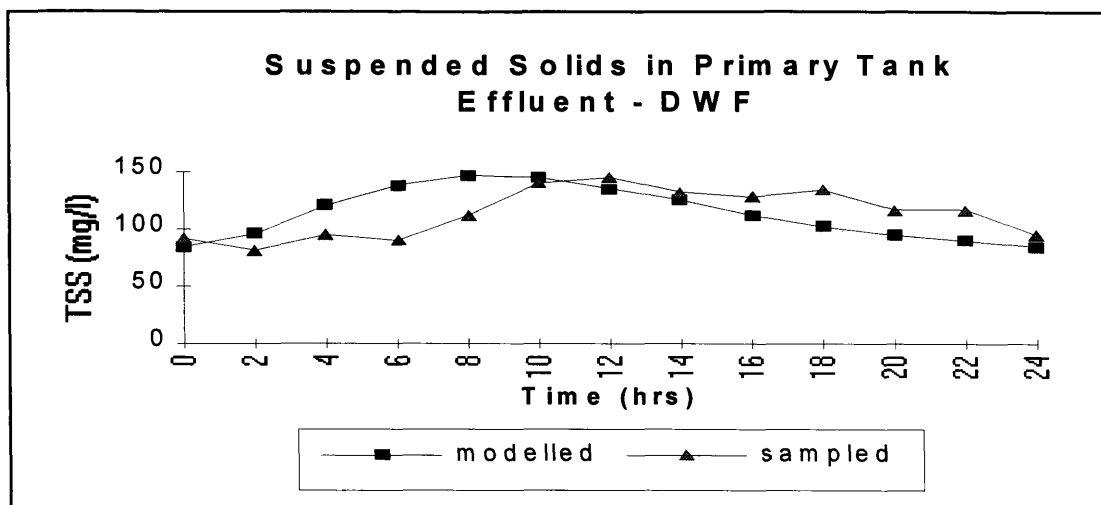


Fig. E.2 BOD in Primary Tank Effluent (STOAT - DWF)

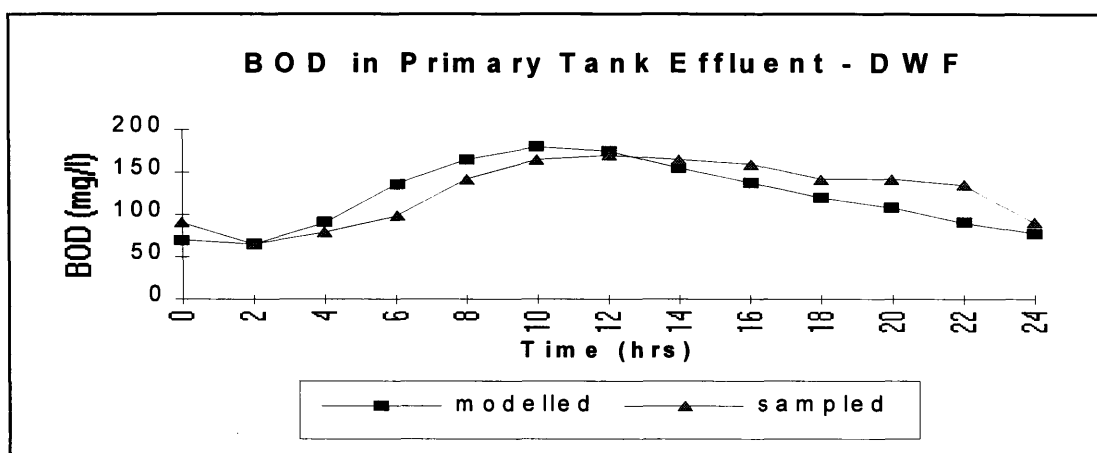


Fig. E.3 Ammonia in Primary Tank Effluent (STOAT - DWF)

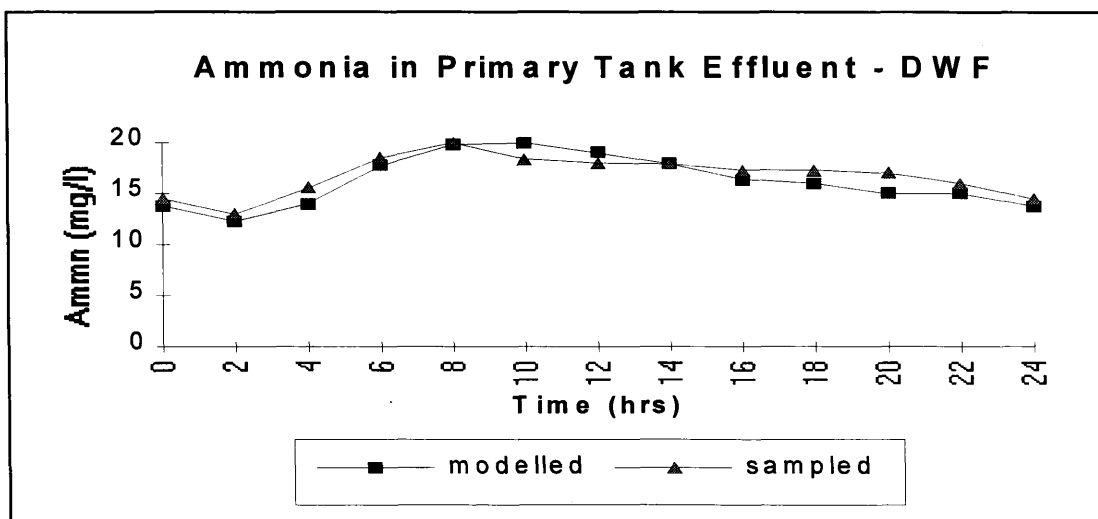


Fig. E.4 TSS in Activated Sludge Effluent (STOAT - DWF)

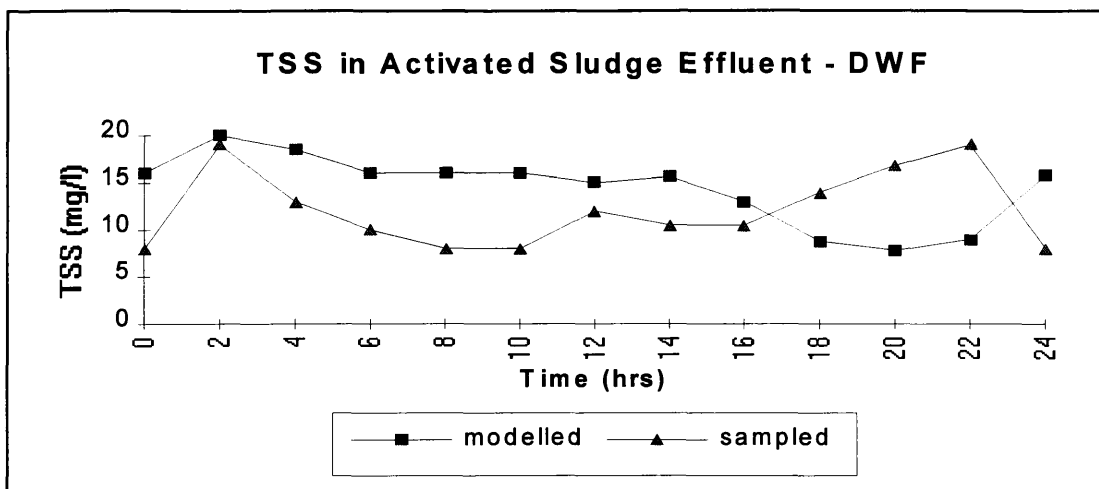


Fig. E.5 BOD in Activated Sludge Effluent (STOAT - DWF)

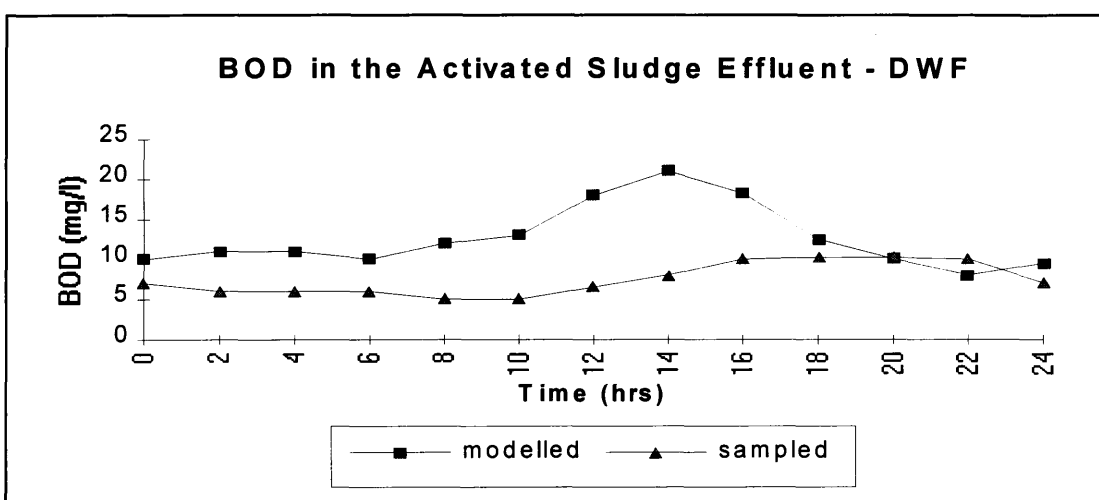


Fig. E.6 Ammonia in Activated Sludge Effluent (STOAT - DWF)

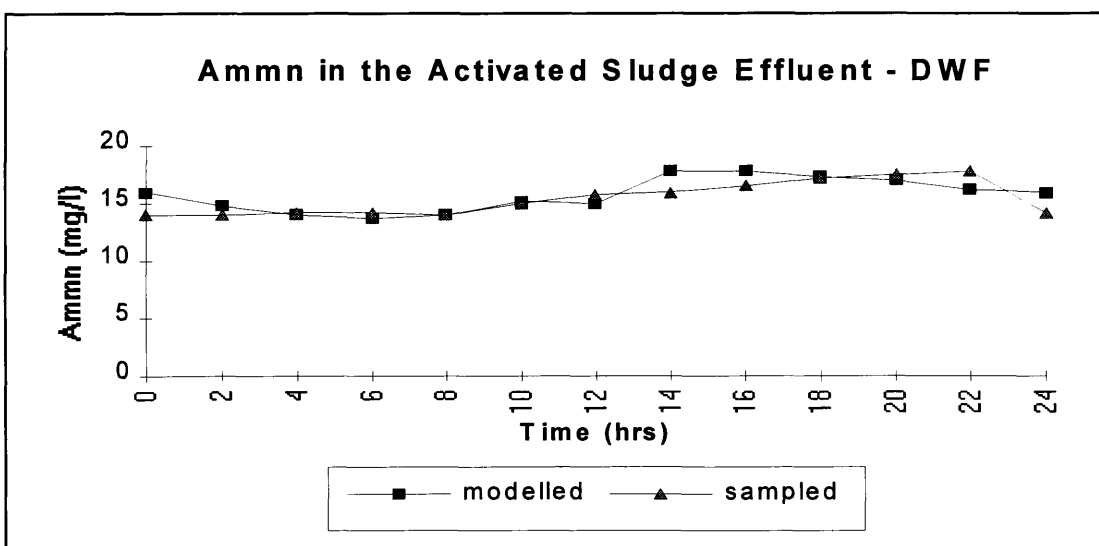


Fig. E.7 TSS in Primary Tank Effluent (STOAT - Storm)

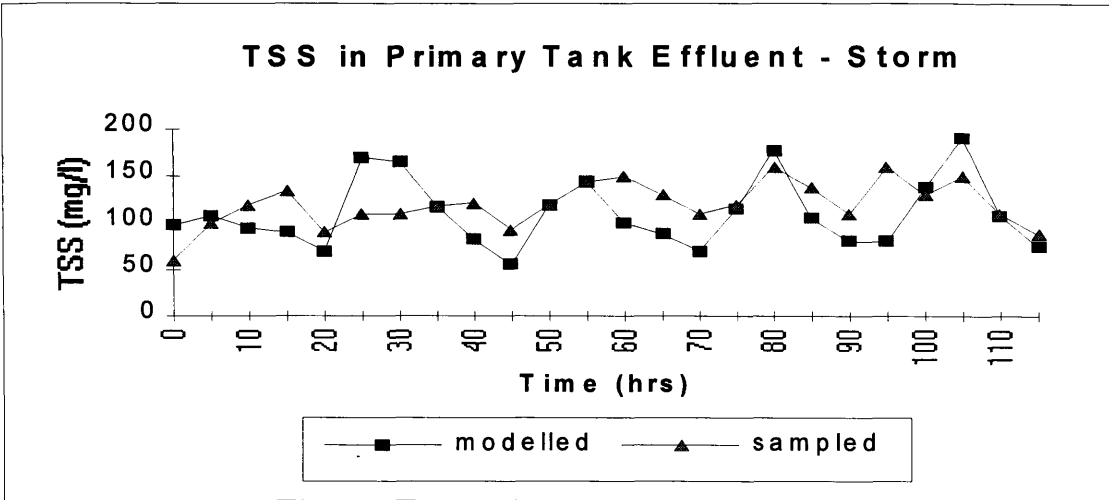


Fig. E.8 BOD in Primary Tank Effluent (STOAT - Storm)

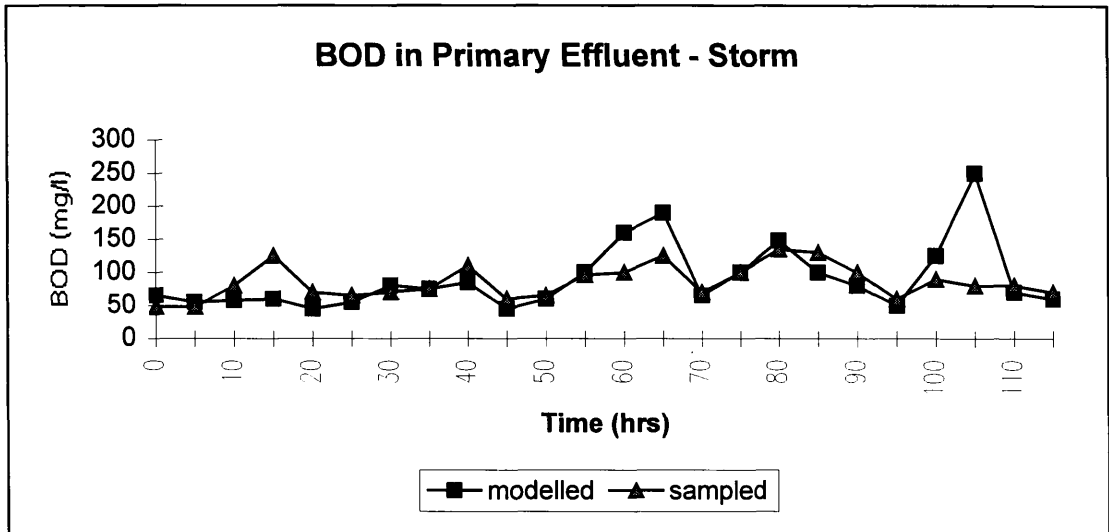


Fig. E.9 Ammonia in Primary Tank Effluent (STOAT - Storm)

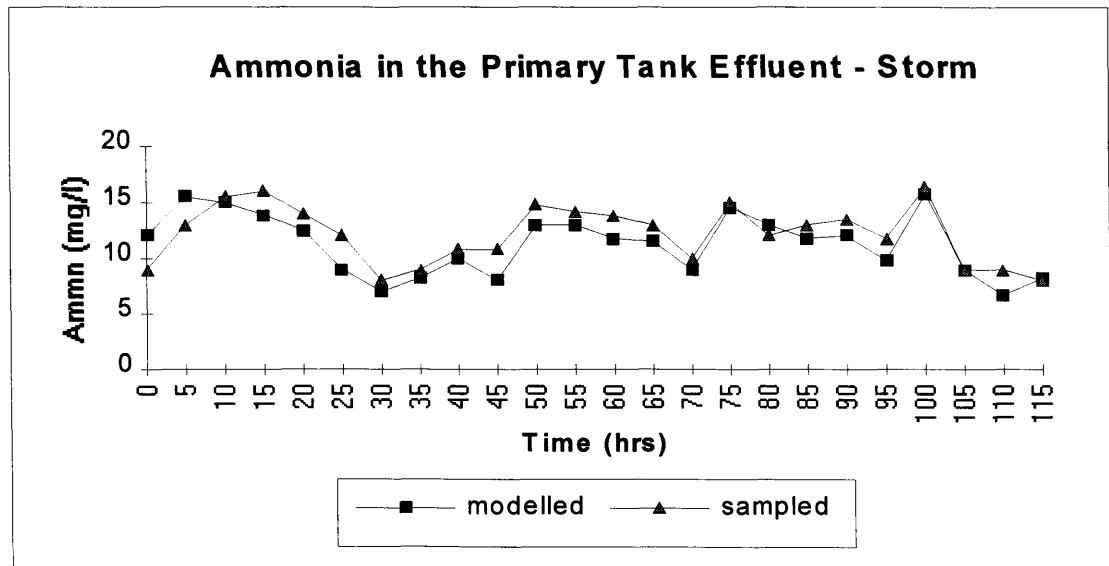


Fig. E.10 TSS in Primary Tank Effluent (GPS-X - DWF)

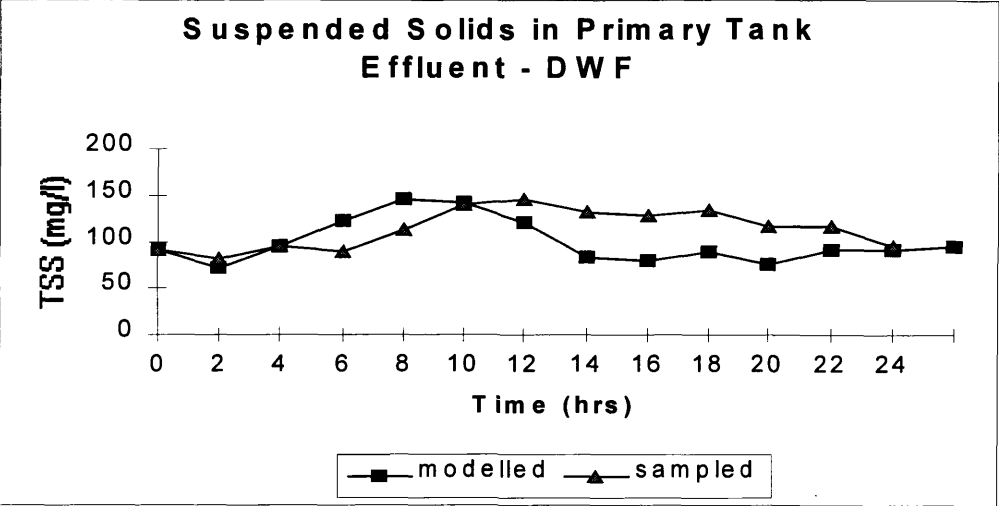


Fig. E.11 BOD in Primary Tank Effluent (GPS-X - DWF)

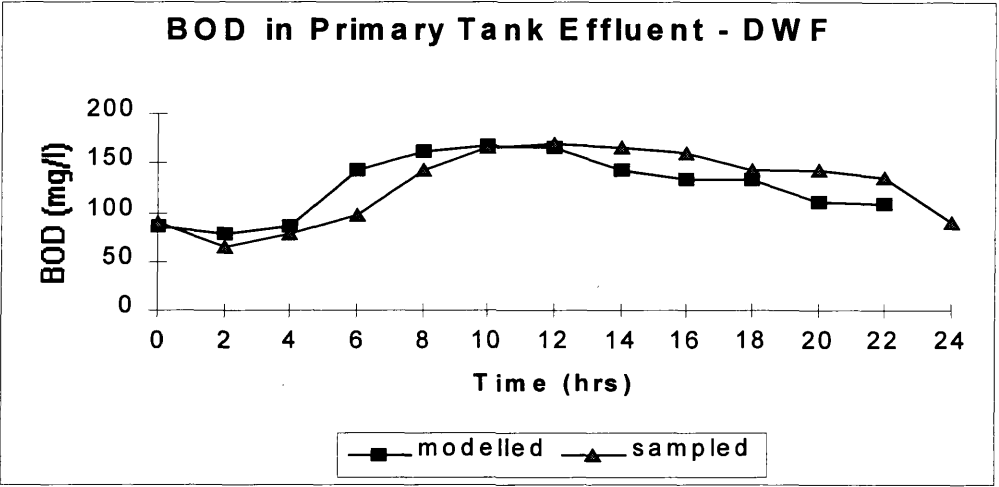


Fig. E.12 Ammonia in Primary Tank Effluent (GPS-X - DWF)

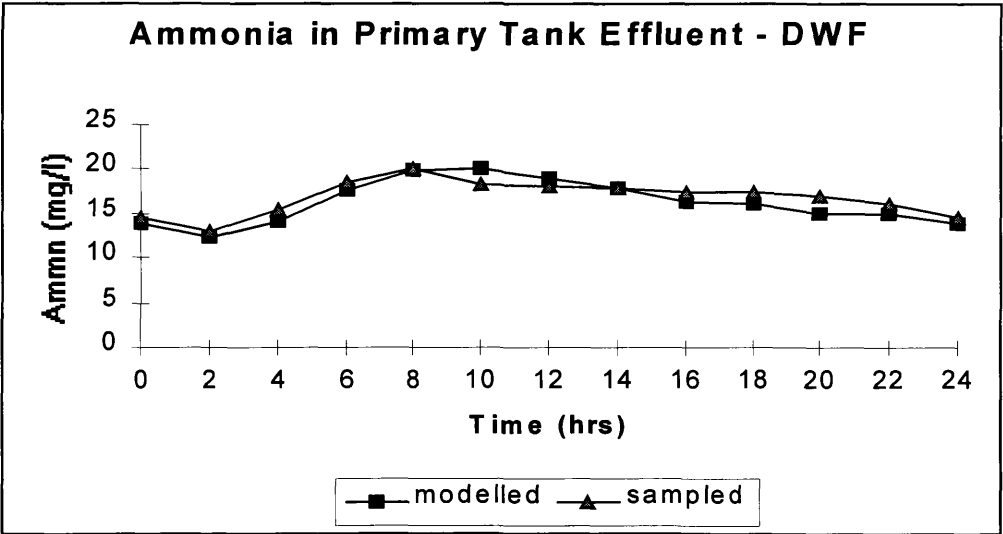


Fig. E.13 TSS in Activated Sludge Effluent (GPS-X - DWF)

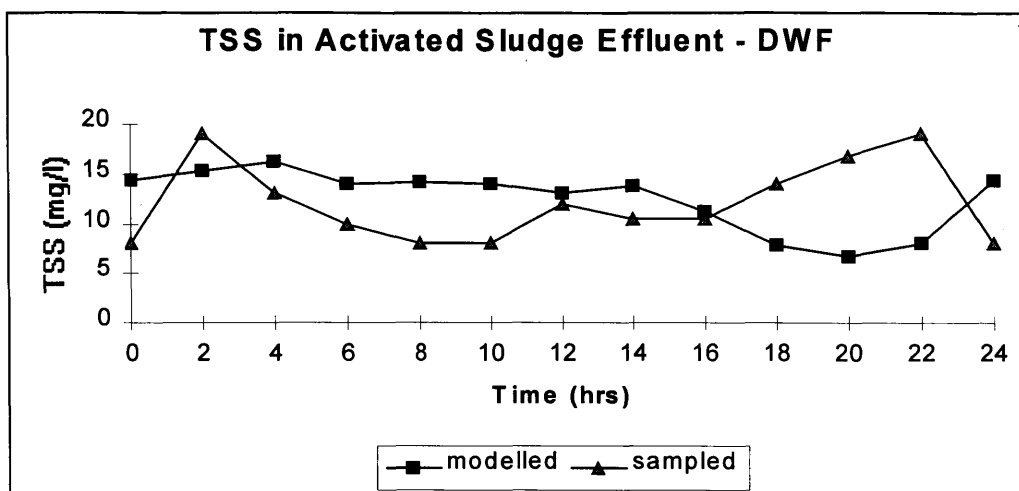


Fig. E.14 BOD in Activated Sludge Effluent (GPS-X - DWF)

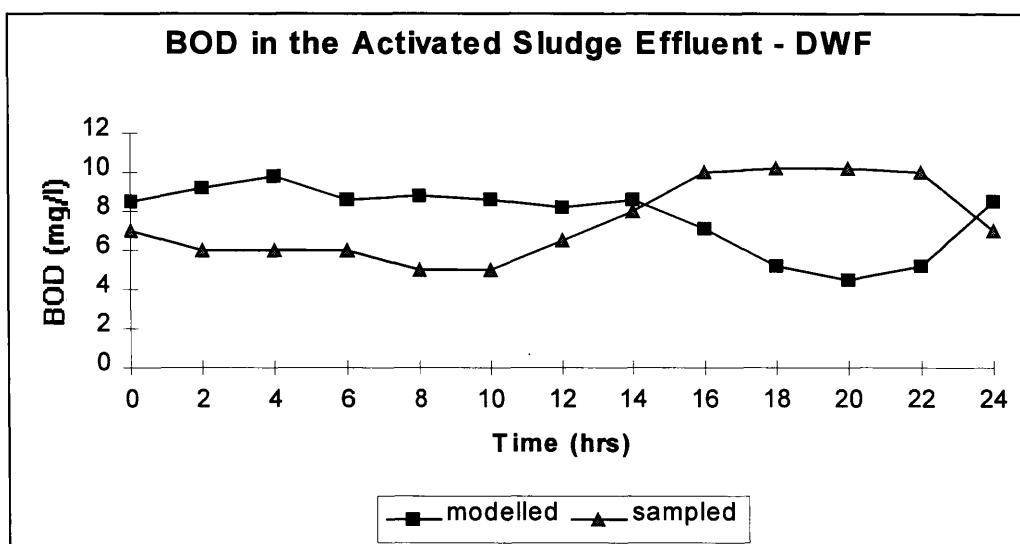


Fig. E.15 Ammonia in Activated Sludge Effluent (GPS-X - DWF)

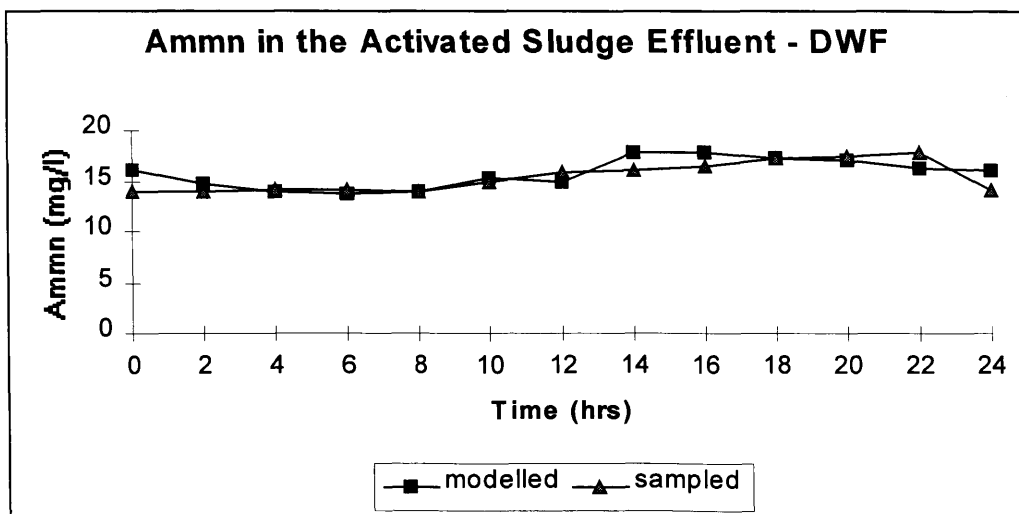


Fig. E.16 TSS in Primary Tank Effluent (GPS-X - Storm)

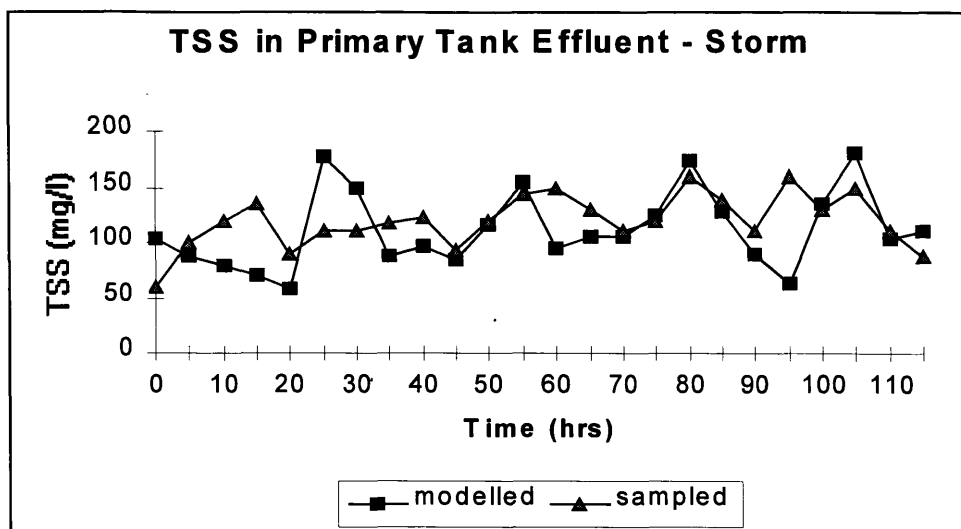


Fig. E.17 BOD in Primary Tank Effluent (GPS-X - Storm)

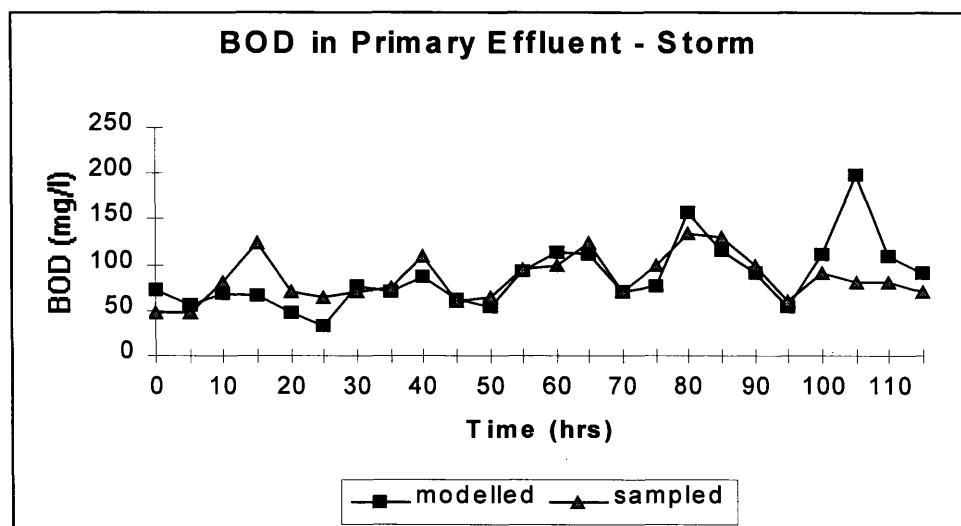


Fig. E.18 Ammonia in Primary Tank Effluent (GPS-X - Storm)

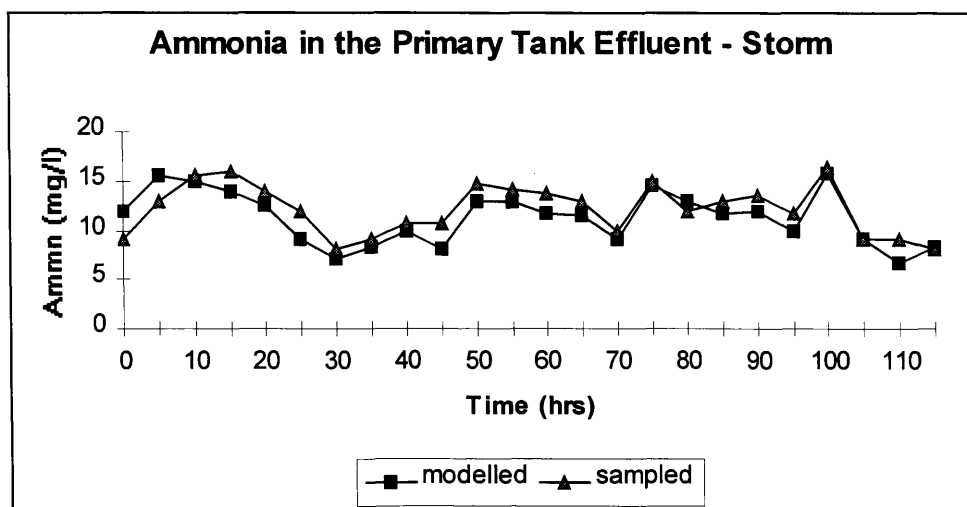


Fig. E.19 TSS in Primary Tank Effluent (STOAT & GPS-X - Storm)

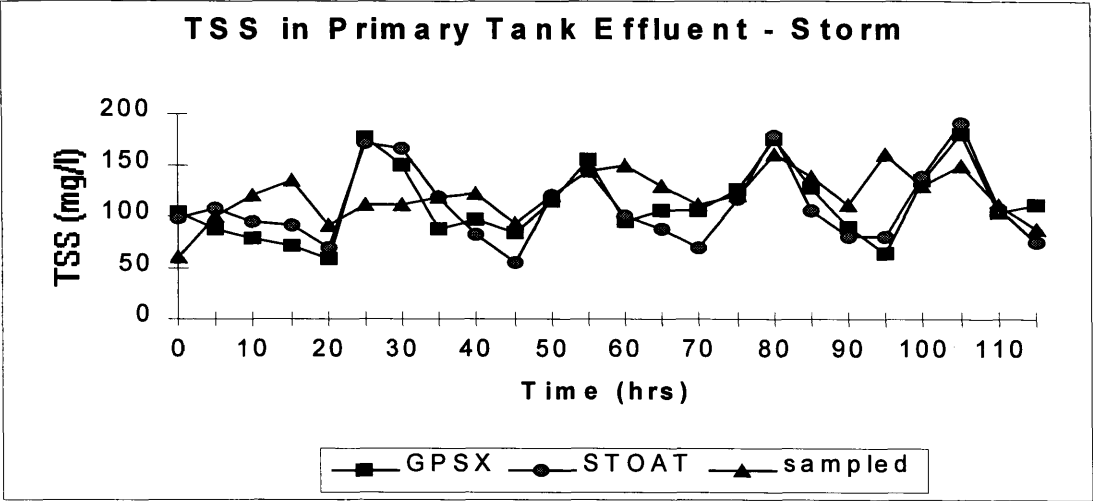


Fig. E.20 BOD in Primary Tank Effluent (STOAT & GPS-X - Storm)

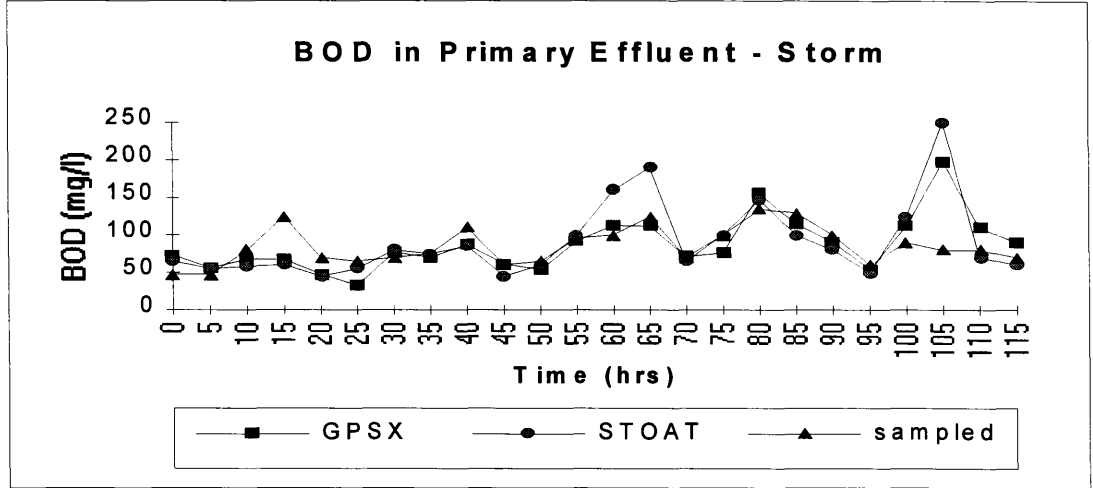


Fig. E.21 Storm Tank Calibrations - Event One - Flow (26/10/95)

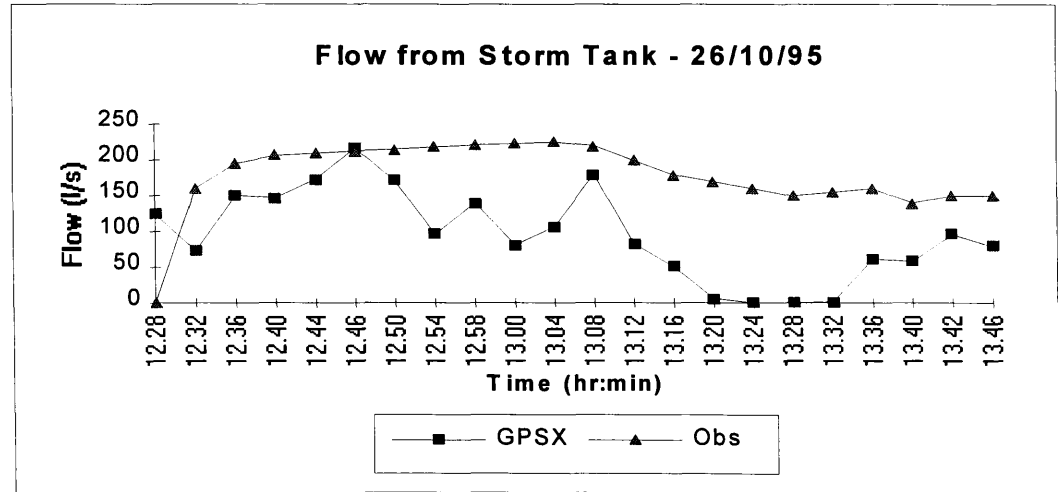


Fig. E.22 Storm Tank Calibrations - Event One - TSS (26/10/95)

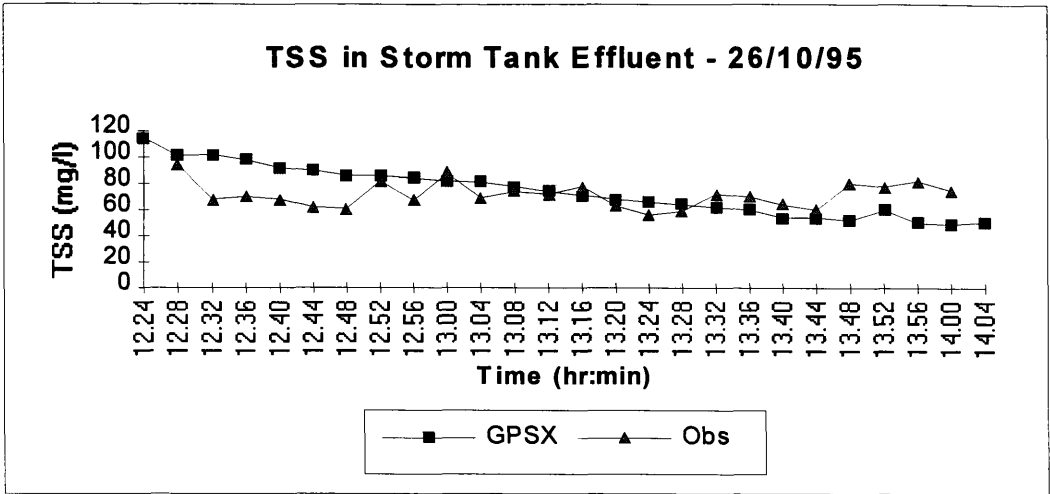


Fig. E.23 Storm Tank Calibrations - Event One - BOD (26/10/95)

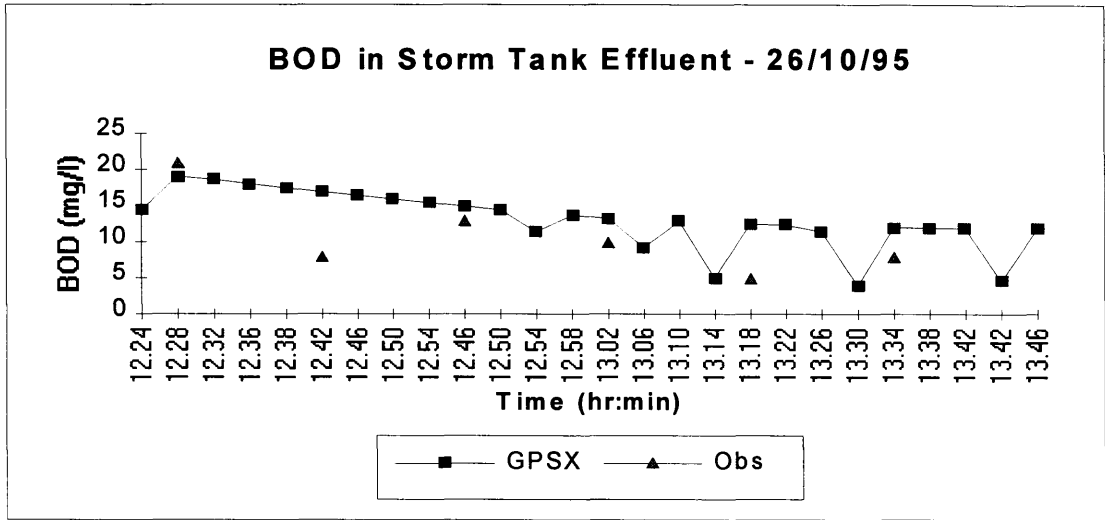


Fig. E.24 Storm Tank Calibrations - Event One - Ammonia (26/10/95)

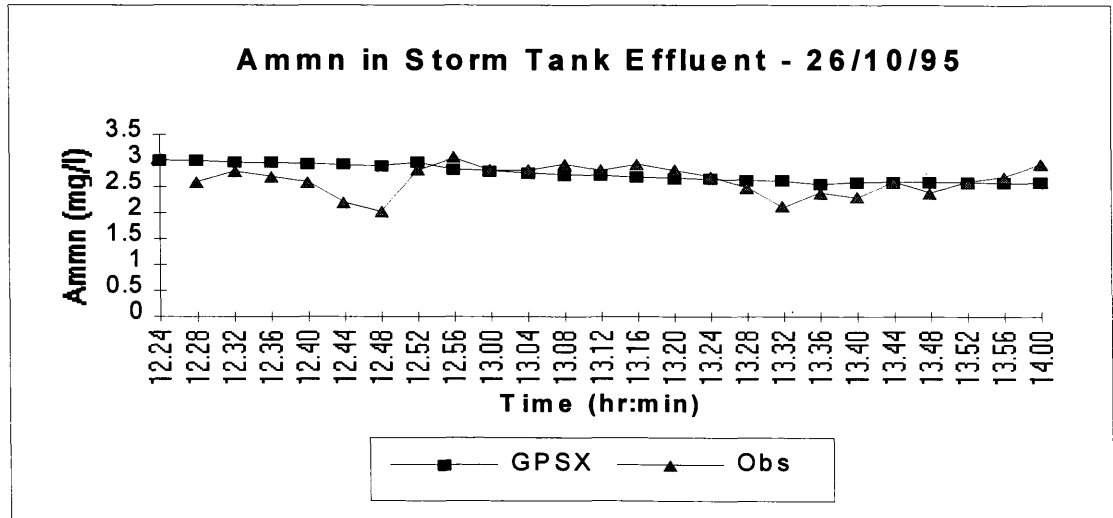


Fig. E.25 Storm Tank Calibrations - Event Two - Flow (31/10/95)

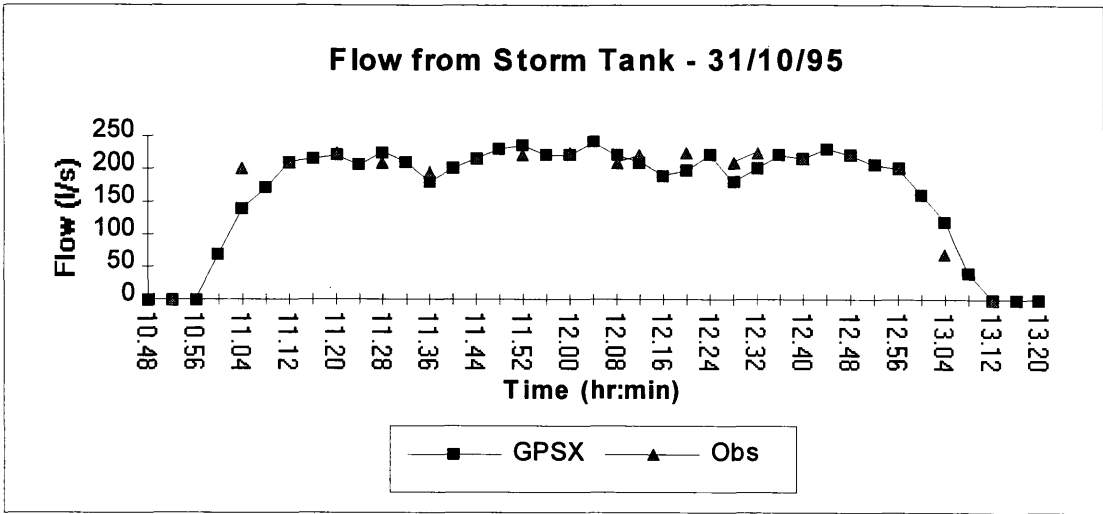


Fig. E.26 Storm Tank Calibrations - Event Two - TSS (31/10/95)

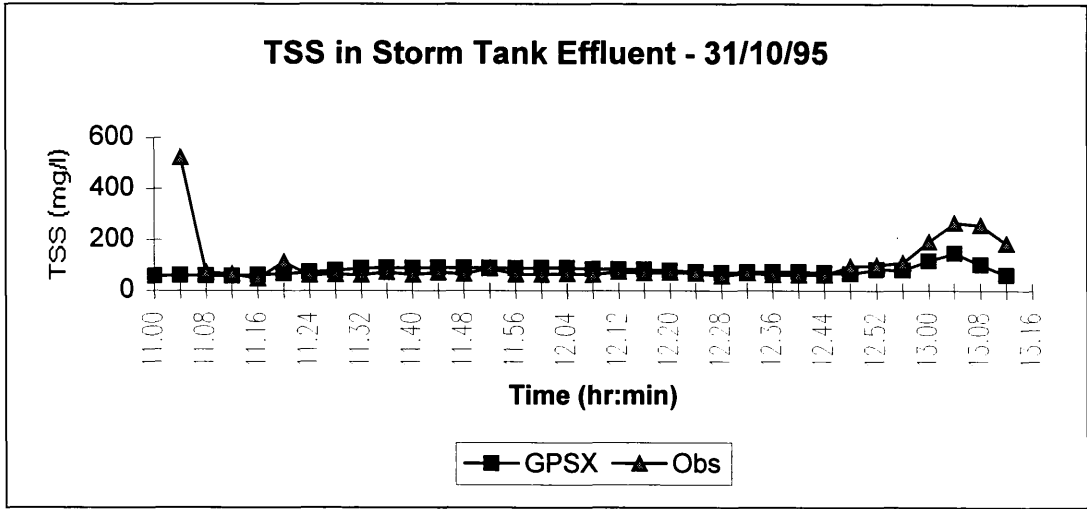


Fig. E.27 Storm Tank Calibrations - Event Two - BOD (31/10/95)

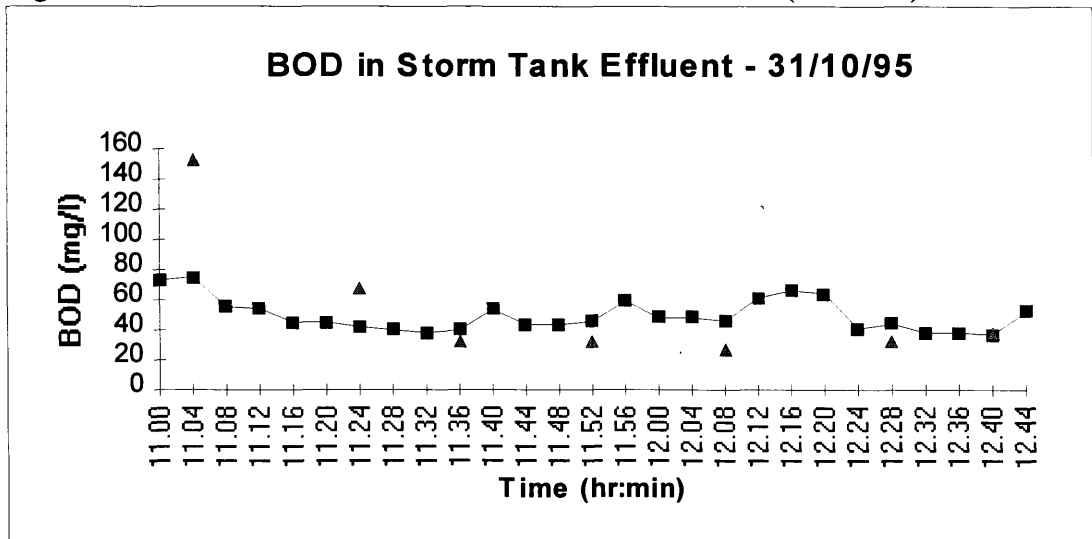


Fig. E.28 Storm Tank Calibrations - Event Two - Ammonia (31/10/95)

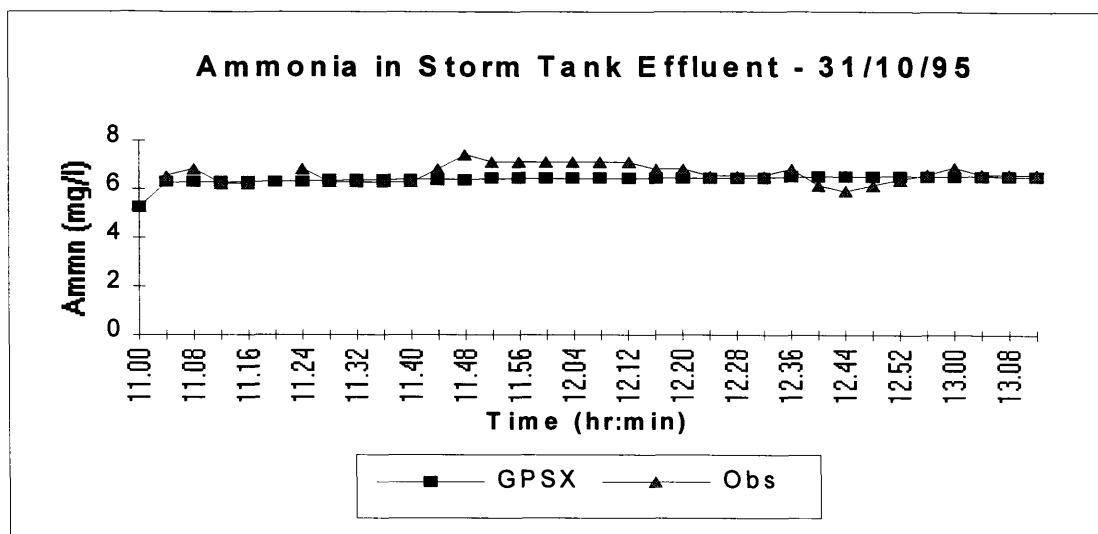


Fig. E.29 WTP Disruptive Period Analysis - Flow

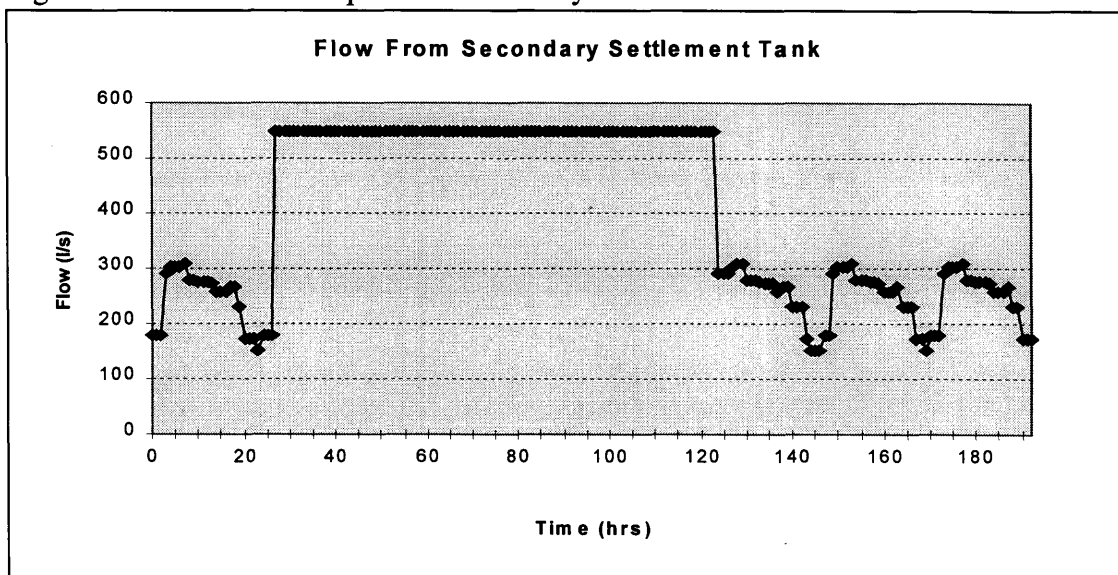


Fig. E.30 WTP Disruptive Period Analysis - TSS and BOD

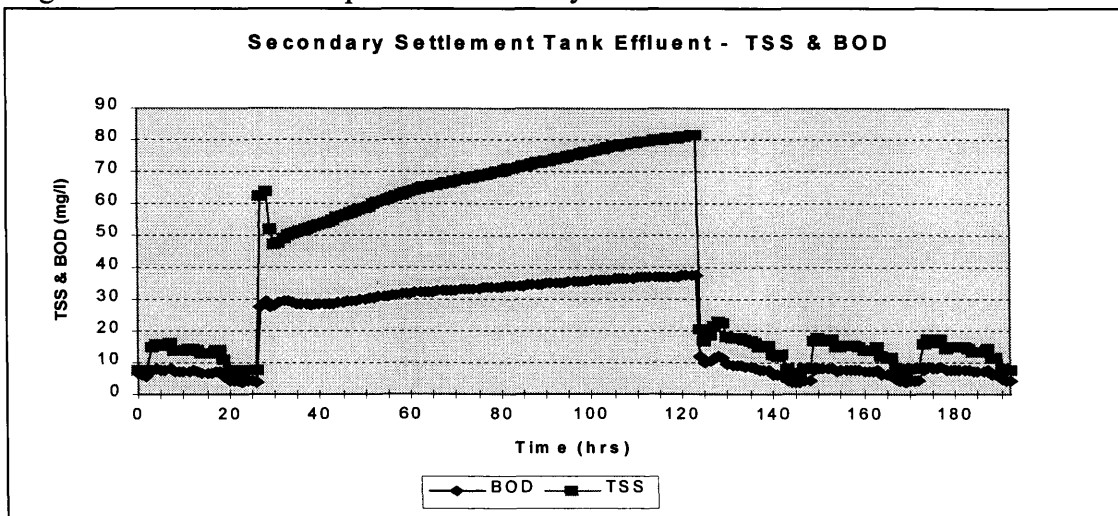


Fig. E.31 Test 1c (Influent of 600l/s for 8 days) – WTP Effluent - Flow

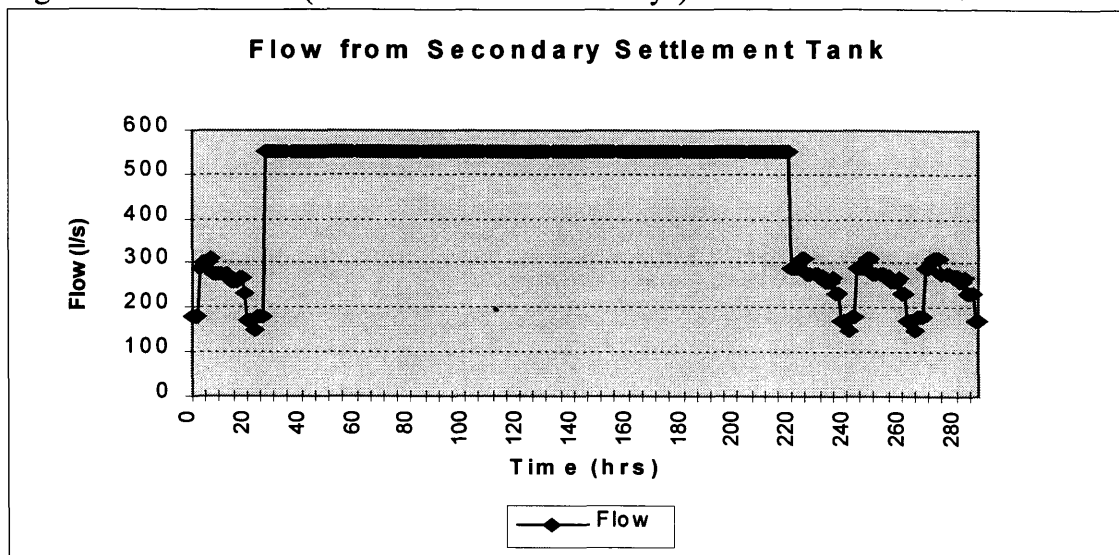


Fig. E.32 Test 1c (Influent of 600l/s for 8 days) – WTP Effluent – TSS & BOD

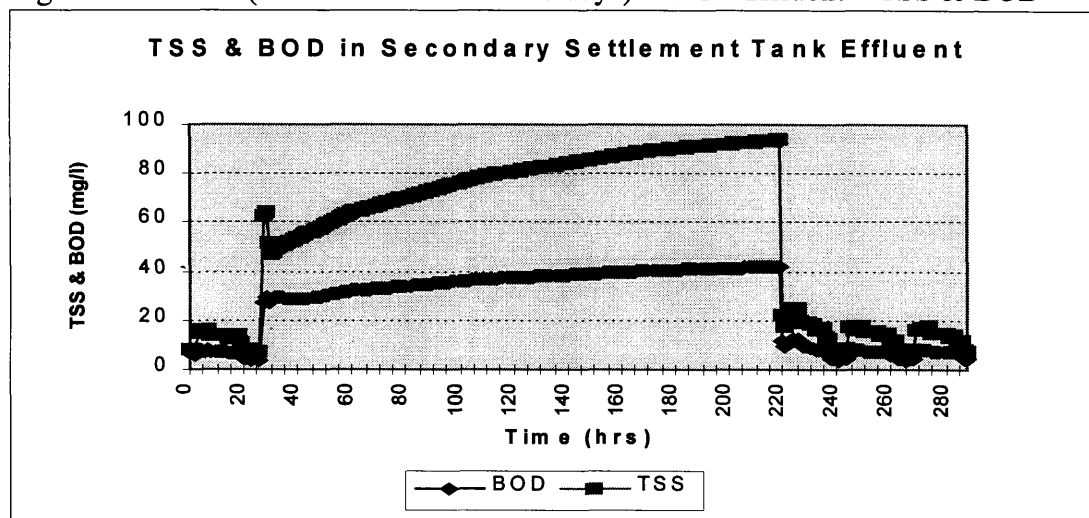


Fig. E.33 Test 1c (Influent of 600l/s for 8 days) – Heterotrophic Biomass in Activated Sludge Tank

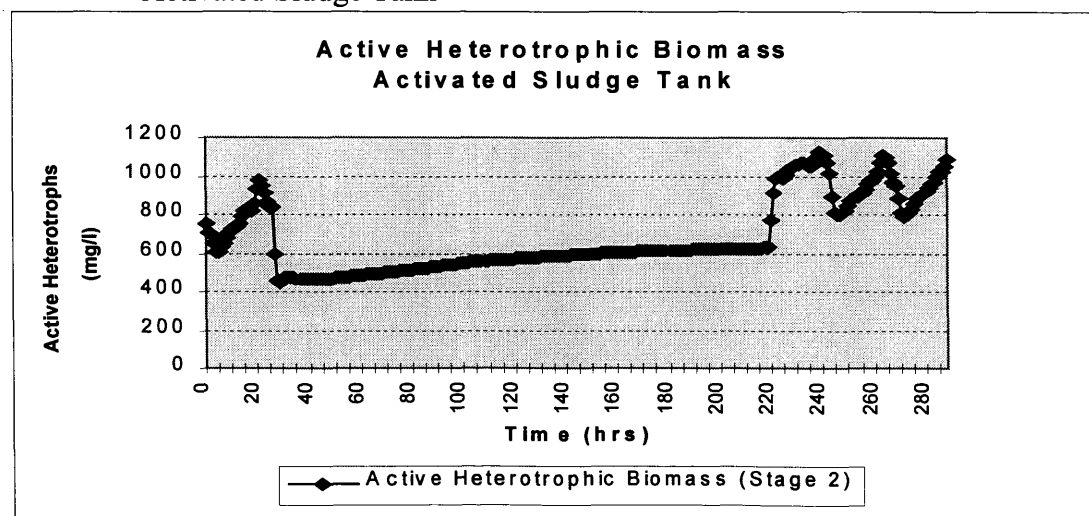


Fig. E.34 Test 2c (Influent of 800l/s for 8 days) – WTP Effluent - Flow

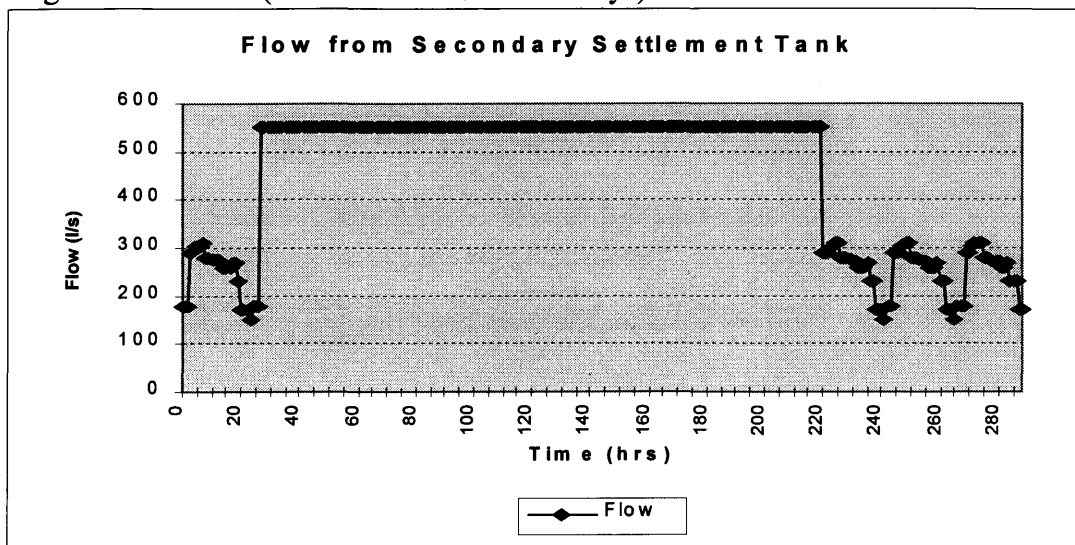


Fig. E.35 Test 2c (Influent of 800l/s for 8 days) – WTP Effluent – TSS & BOD

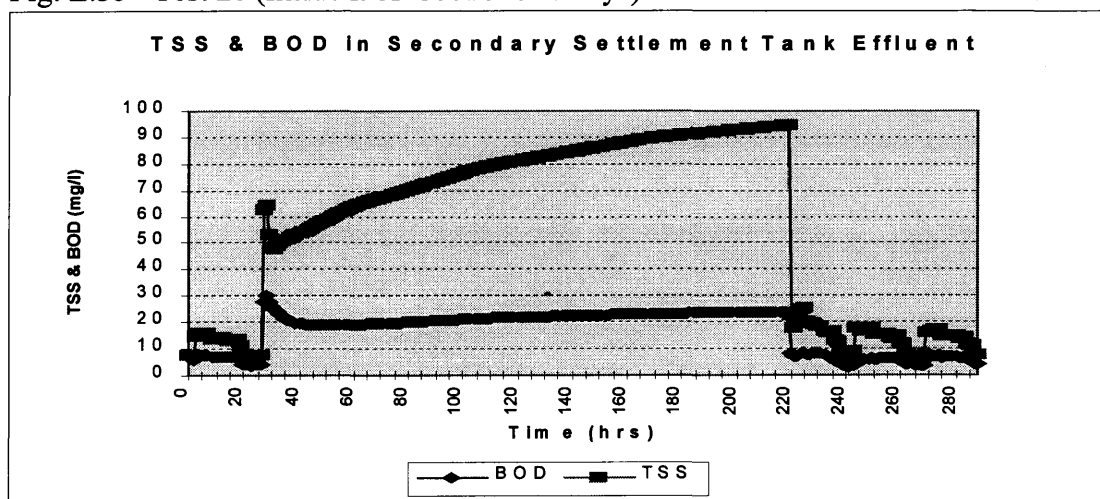


Fig. E.36 Test 2c (Influent of 800l/s for 8 days) – Heterotrophic Biomass in Activated Sludge Tank

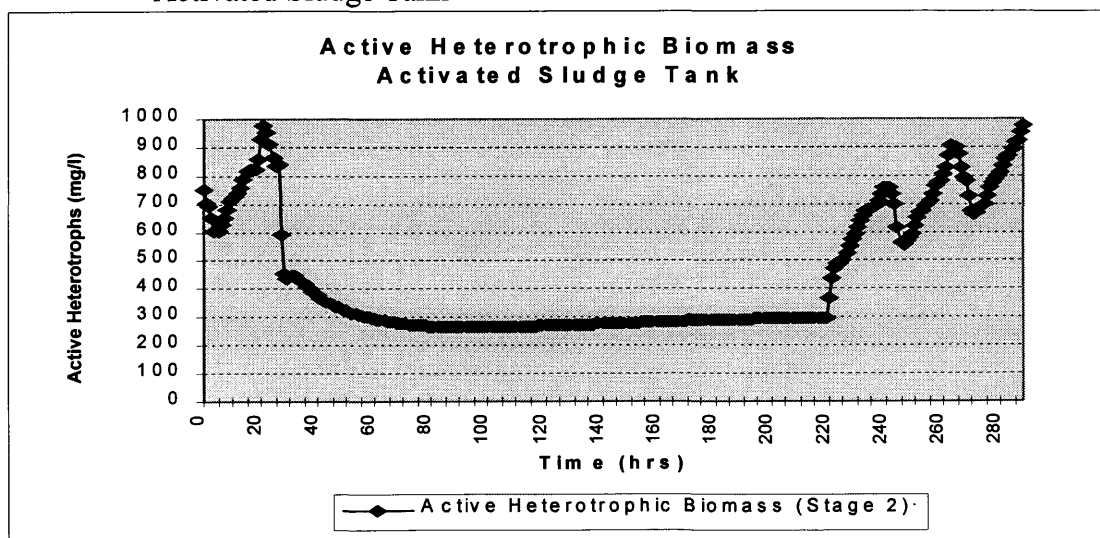


Fig. E.37 Flow from Secondary Settlement Tank - Sludge Loss Analysis

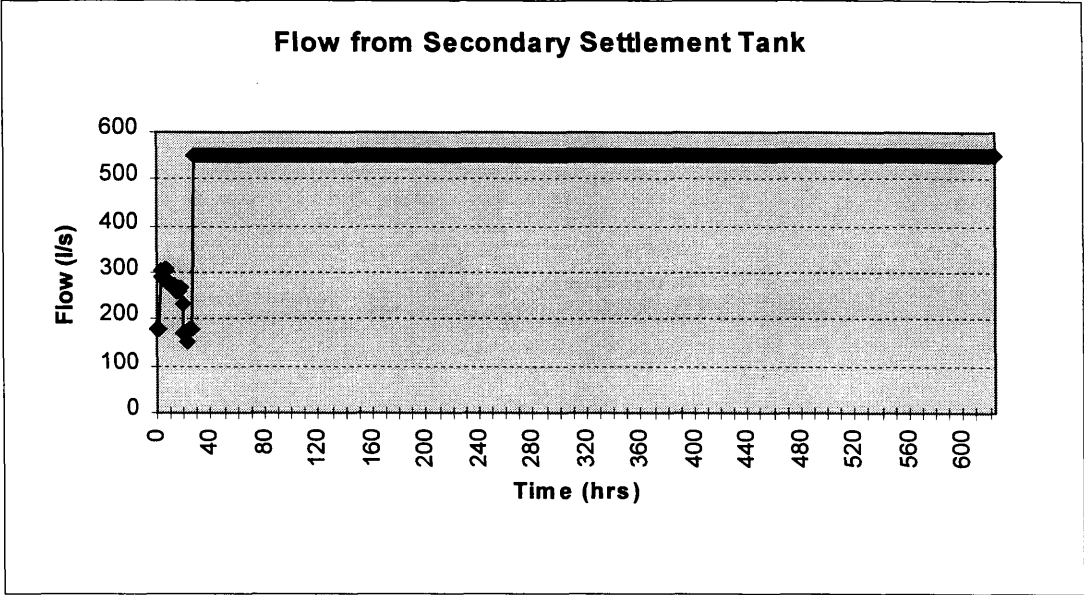


Fig. E.38 TSS & BOD in from Secondary Settlement Tank - Sludge Loss Analysis

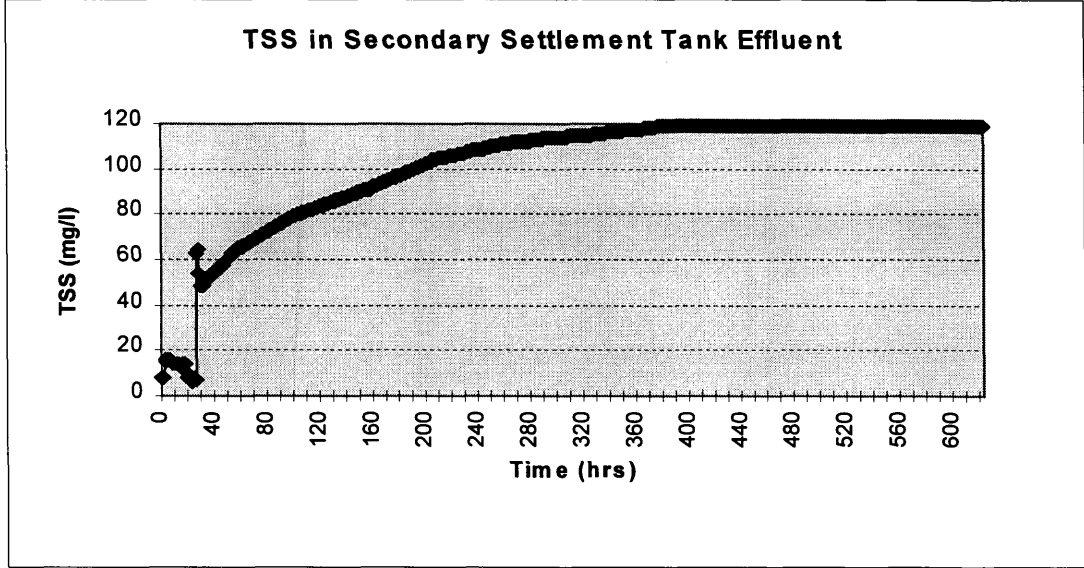


Fig. E.39 Test 3a (Influent of 1000l/s for 4 days) – WTP Effluent - Flow

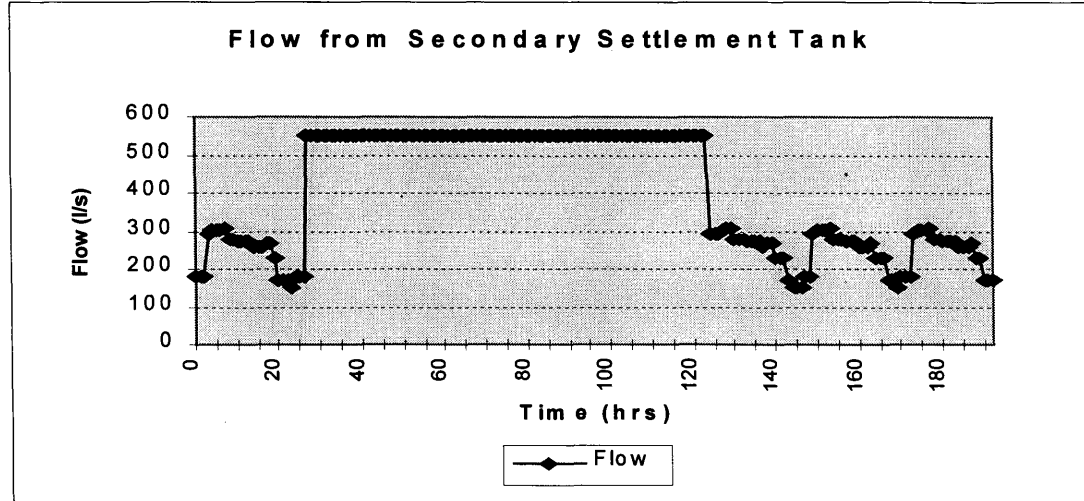


Fig. E.40 Test 3a (Influent of 1000l/s for 4 days) – WTP Effluent – TSS & BOD

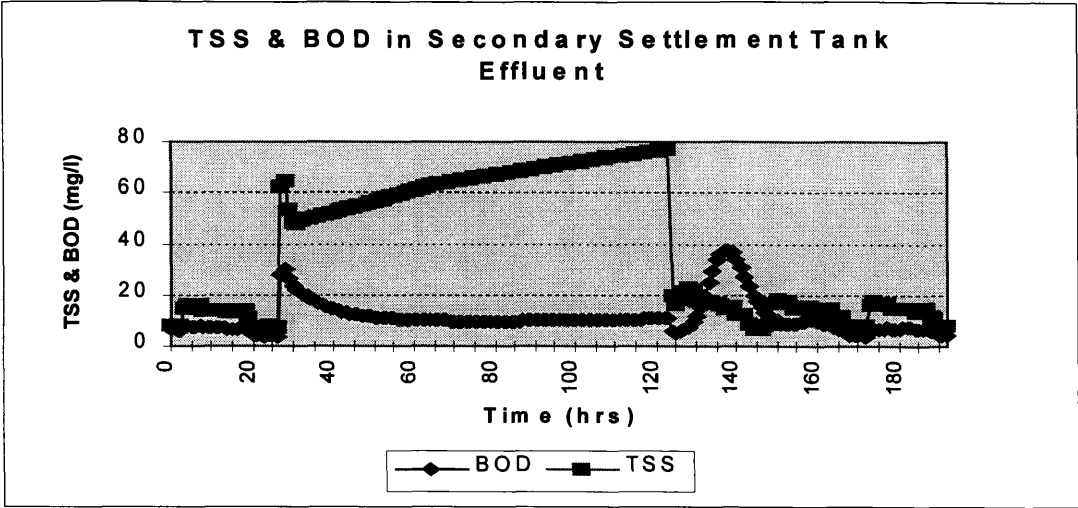


Fig. E.41 Test 3a (Influent of 1000l/s for 4 days) Heterotrophic Biomass in Activated Sludge Tank

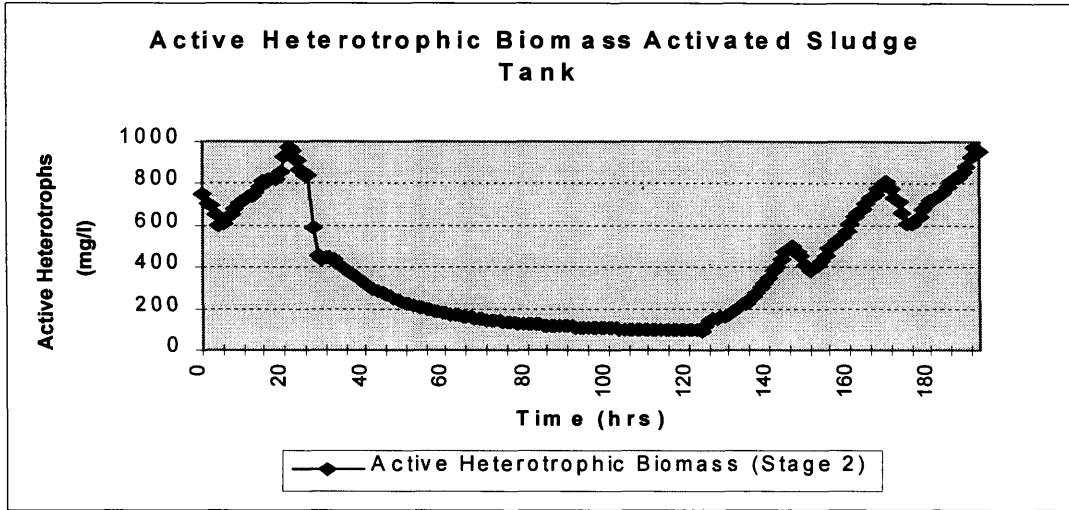


Fig. E.42 Test 3b (Influent of 1000l/s for 6 days) – WTP Effluent - Flow

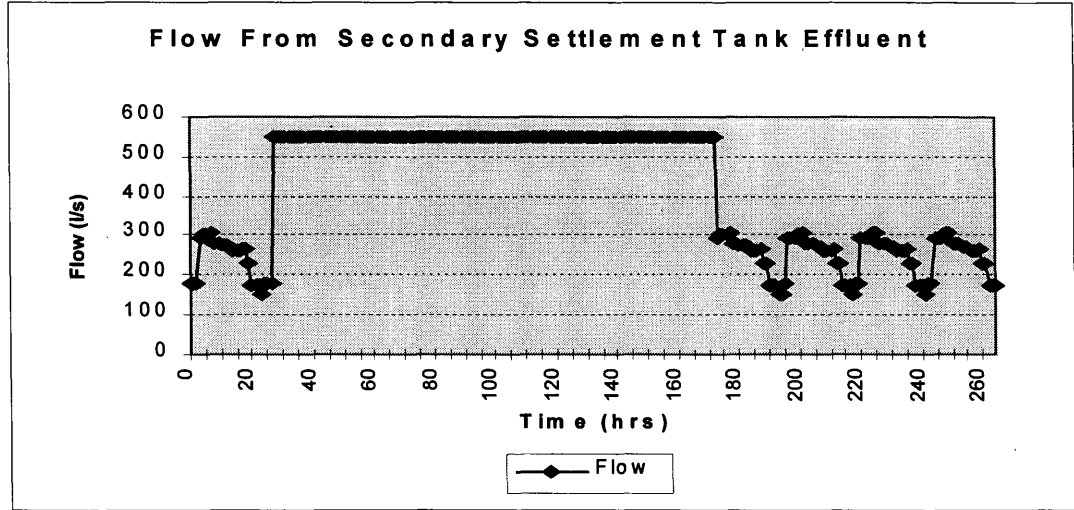


Fig. E.43 Test 3b (Influent of 1000l/s for 6 days) – WTP Effluent – TSS & BOD

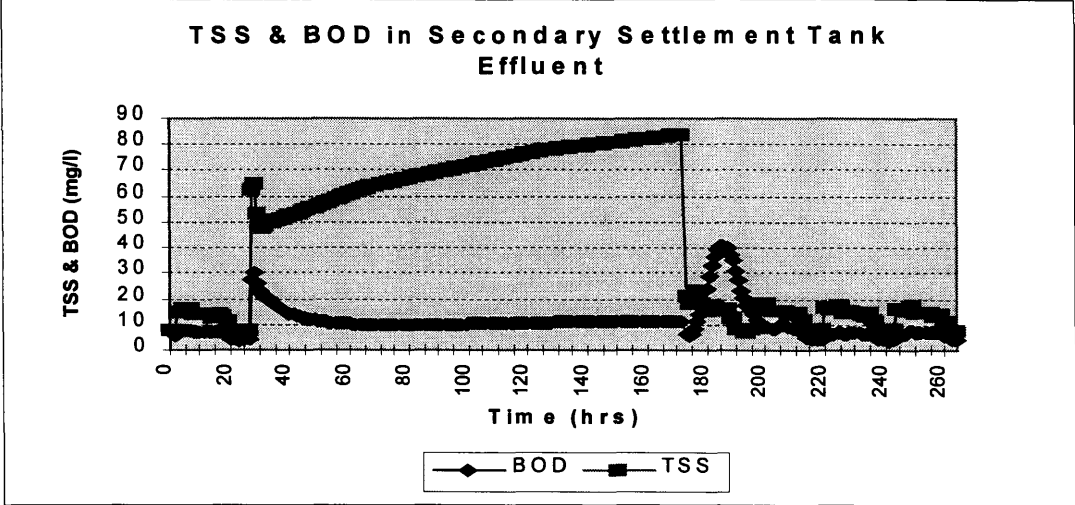


Fig. E.44 Test 3b (Influent of 1000l/s for 6 days) – Heterotrophic Biomass in Activated Sludge Tank

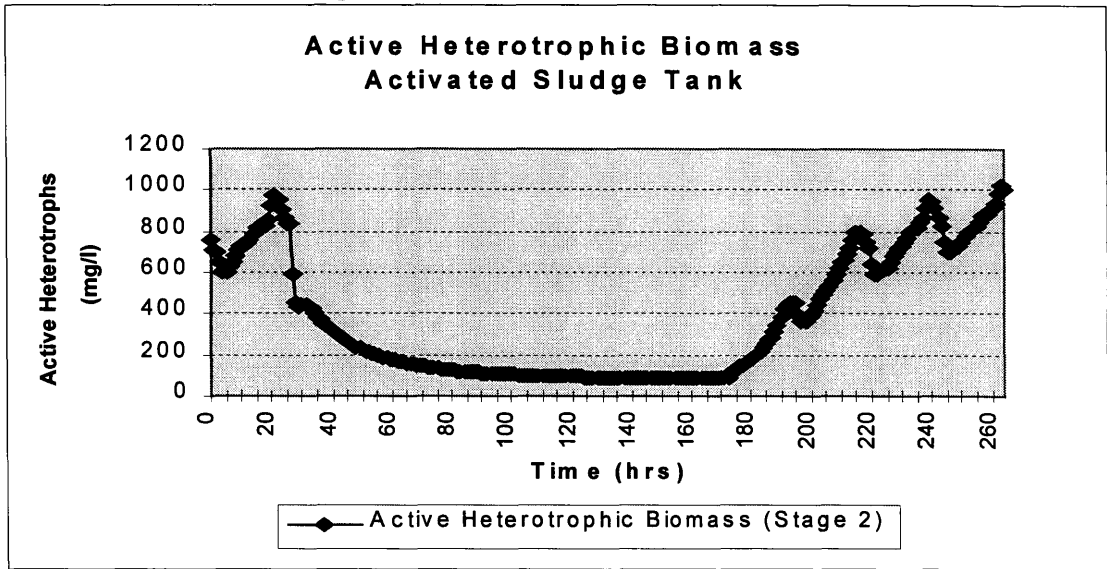


Fig. E.45 Test 3c (Influent of 1000l/s for 8 days) – WTP Effluent - Flow

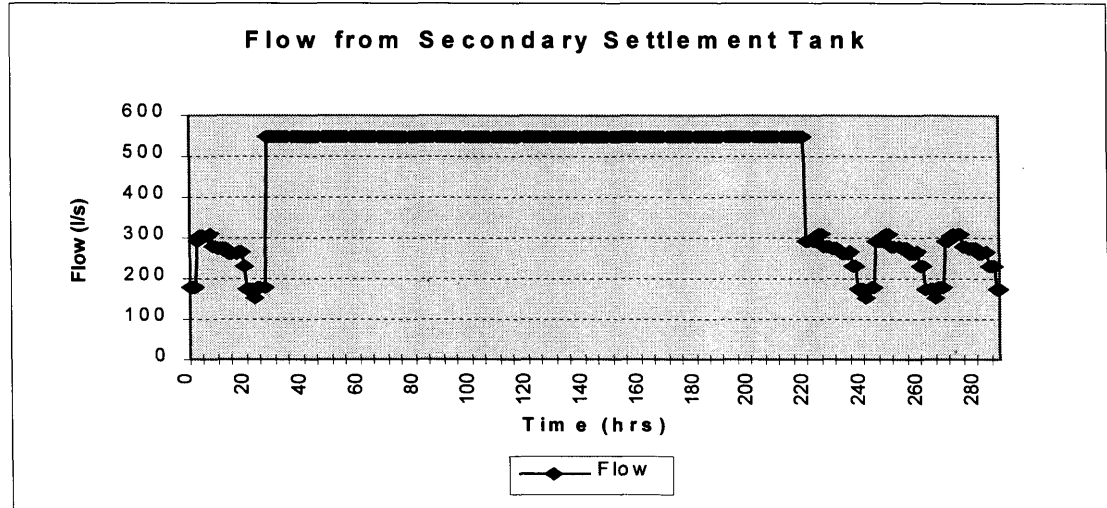


Fig. E.46 Test 3c (Influent of 1000l/s for 8 days) – WTP Effluent – TSS & BOD

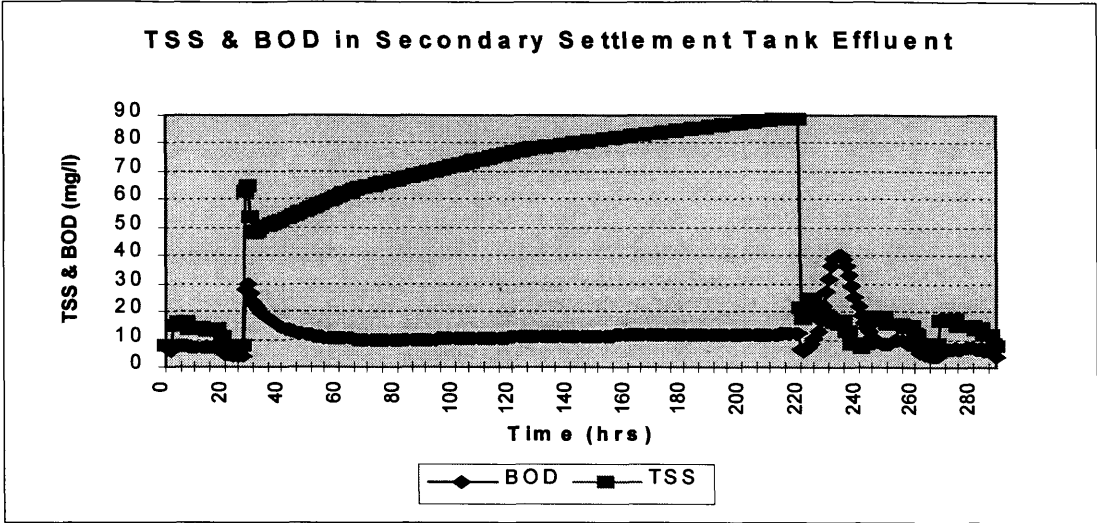


Fig. E.47 Test 3c (Influent of 1000l/s for 8 days) Heterotrophic Biomass in Activated Sludge Tank

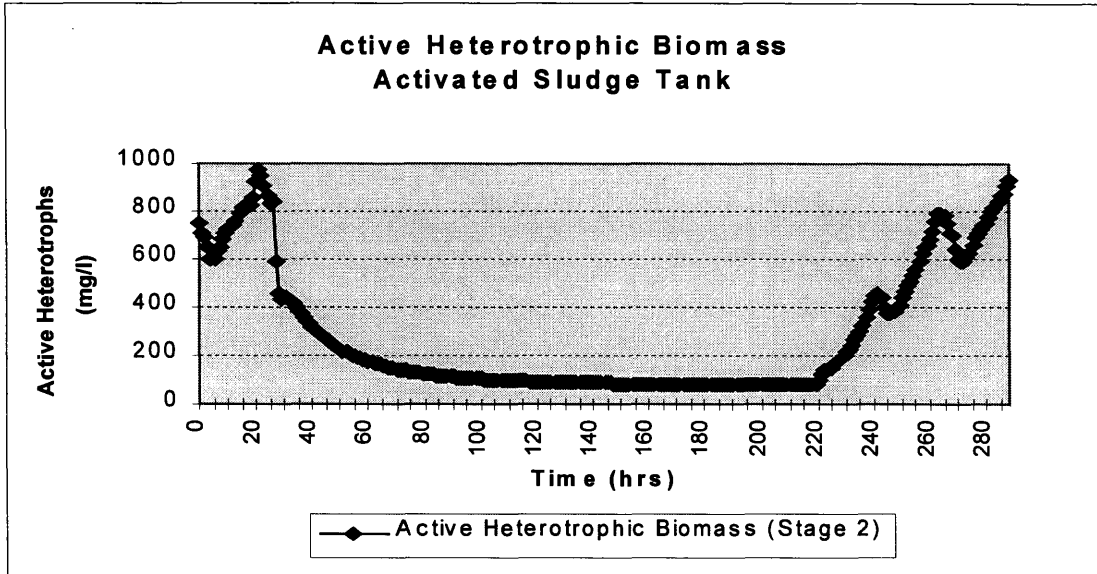


Fig. E.48 Influent of 1000l/s for 1 day - WTP Effluent - Flow

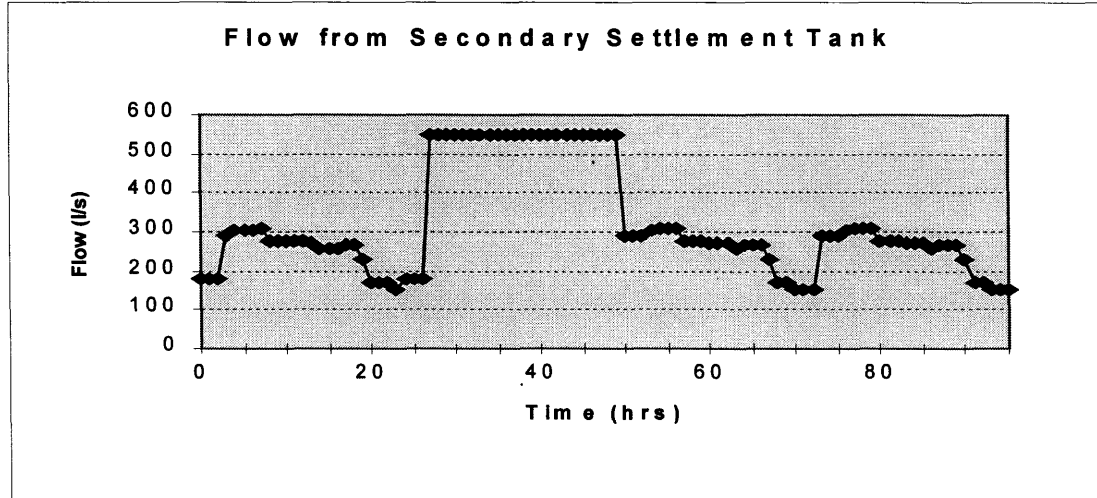


Fig. E.49 Influent of 1000l/s for 1 day - WTP Effluent - BOD

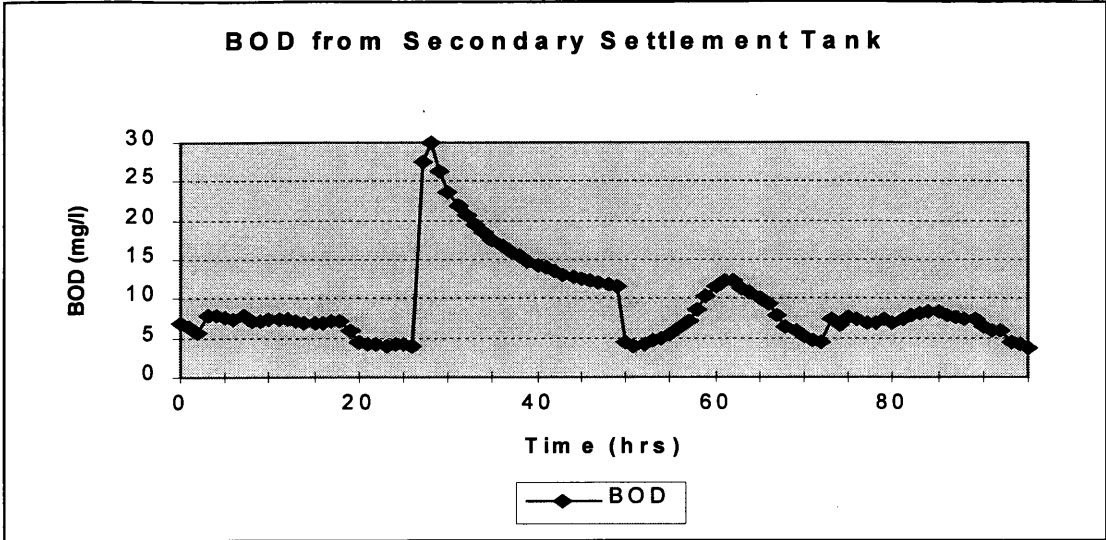


Fig. E.50 Influent of 1000l/s for 1 day - Heterotrophic Biomass in Activated Sludge Tank

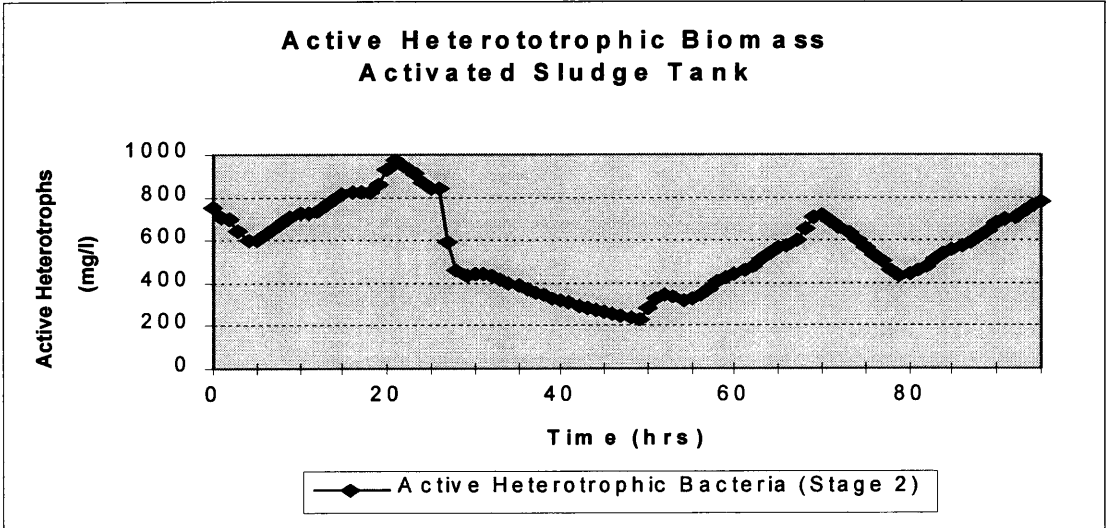


Fig. E.51 Influent of 1000l/s for 2 days - WTP Effluent - Flow

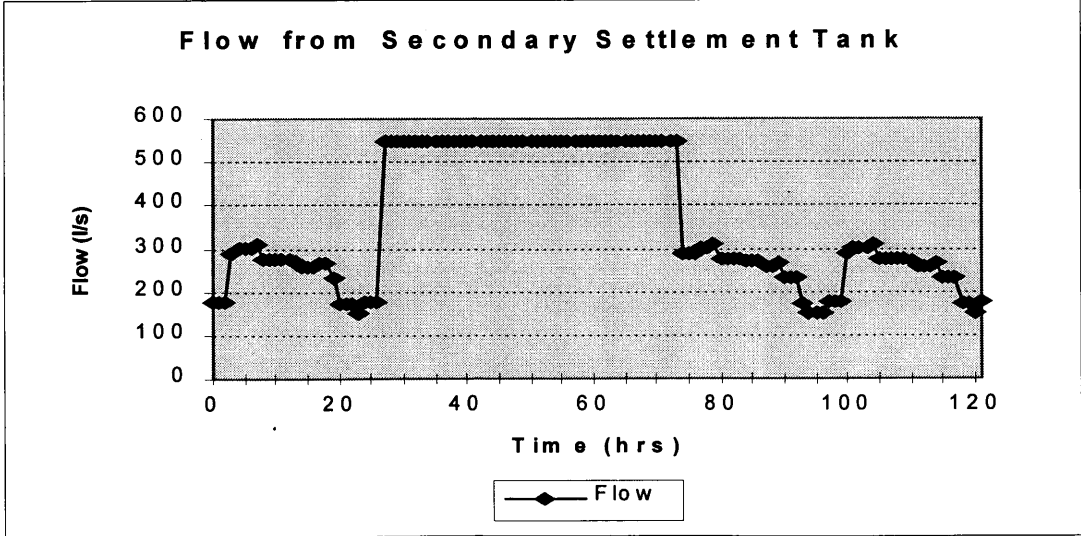


Fig. E.52 Influent of 1000l/s for 2 days - WTP Effluent - BOD

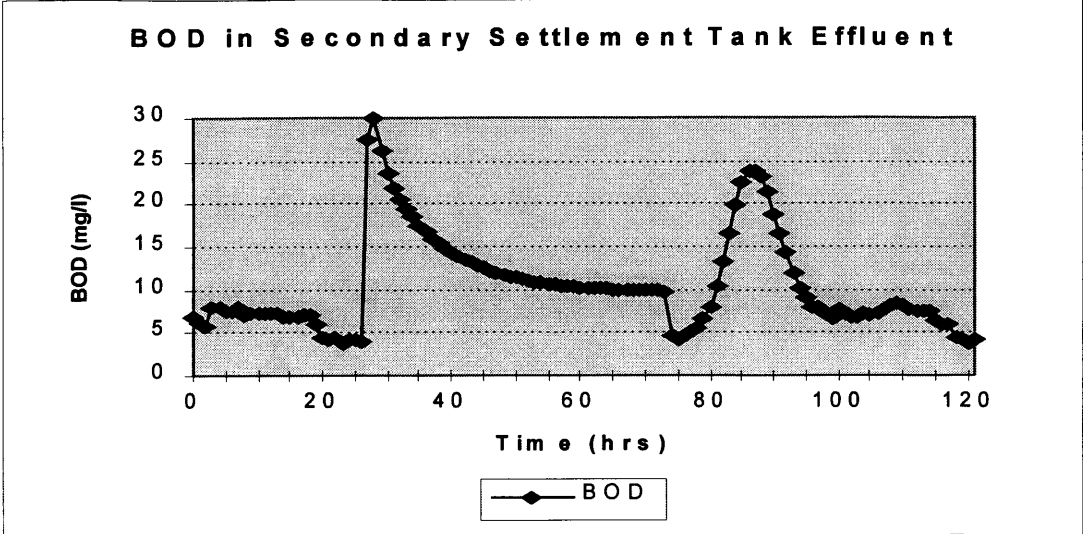


Fig. E.53 Influent of 1000l/s for 2 days - Heterotrophic Biomass in Activated Sludge Tank

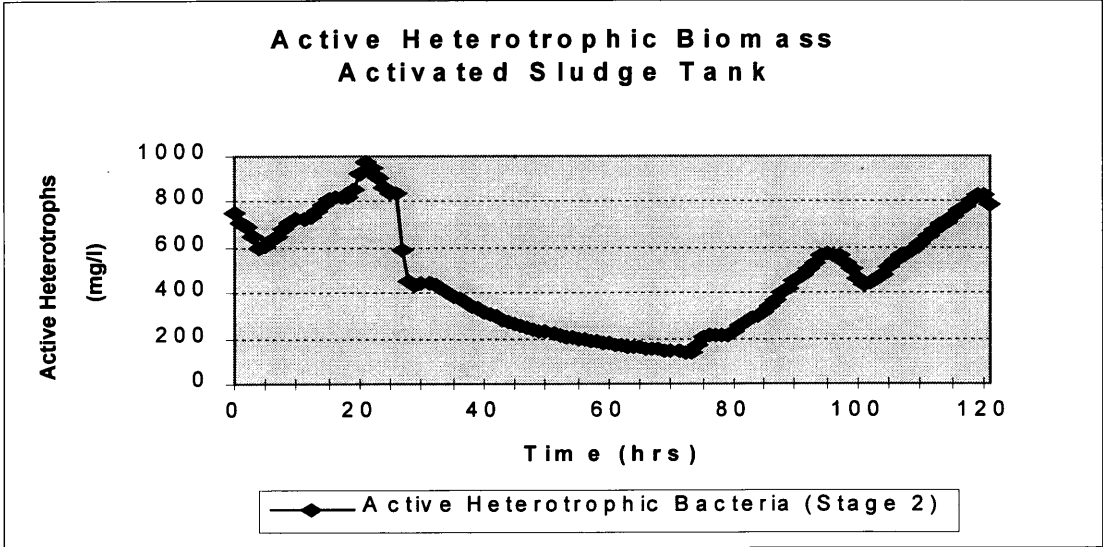


Fig. E.54 Influent of 1000l/s for 25 days - WTP Effluent - Flow

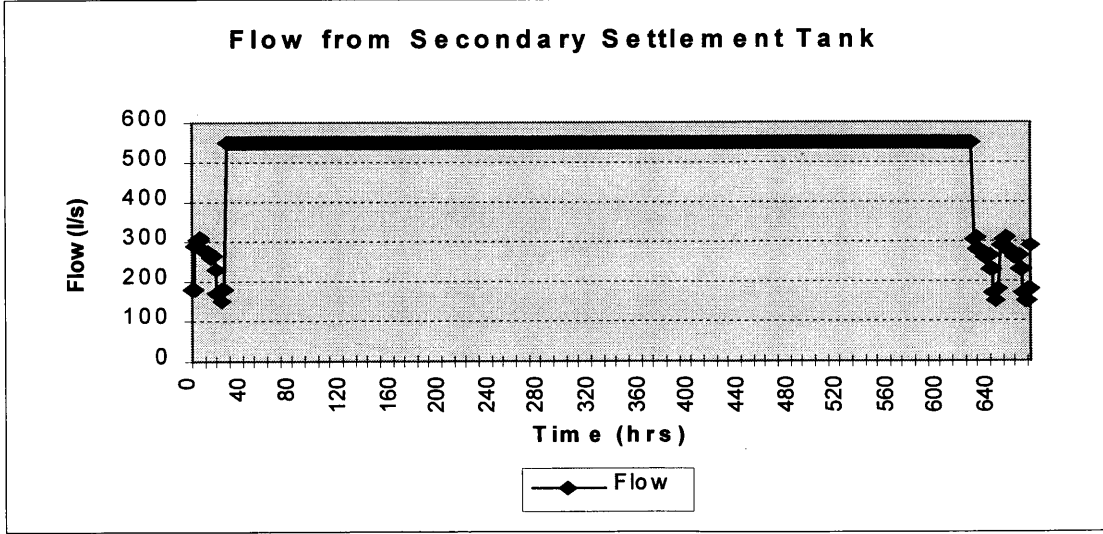


Fig. E.55 Influent of 1000l/s for 25 days - WTP Effluent - BOD

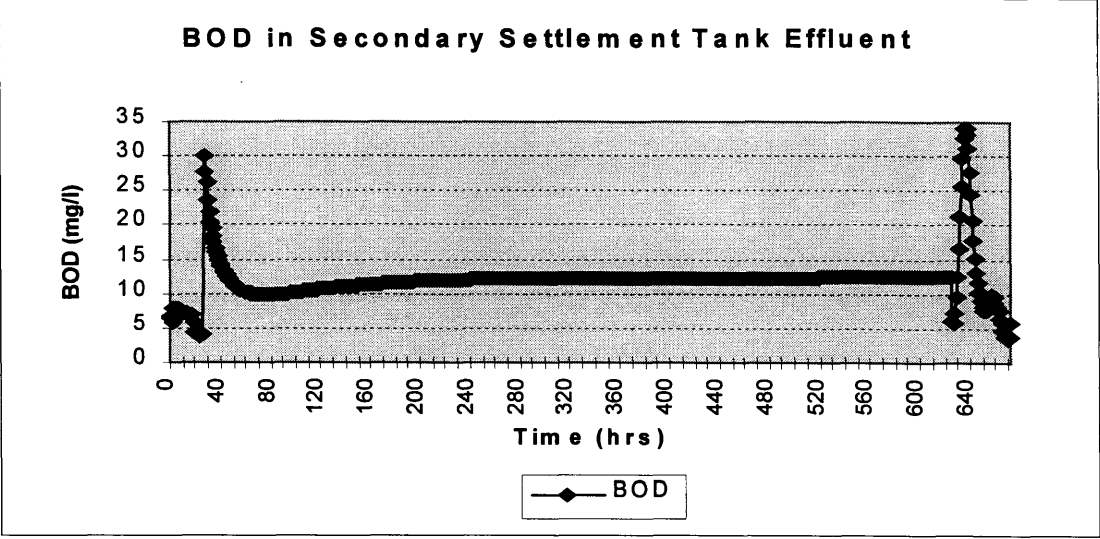


Fig. E.56 Influent of 1000l/s for 25 days - Heterotrophic Biomass in Activated Sludge Tank

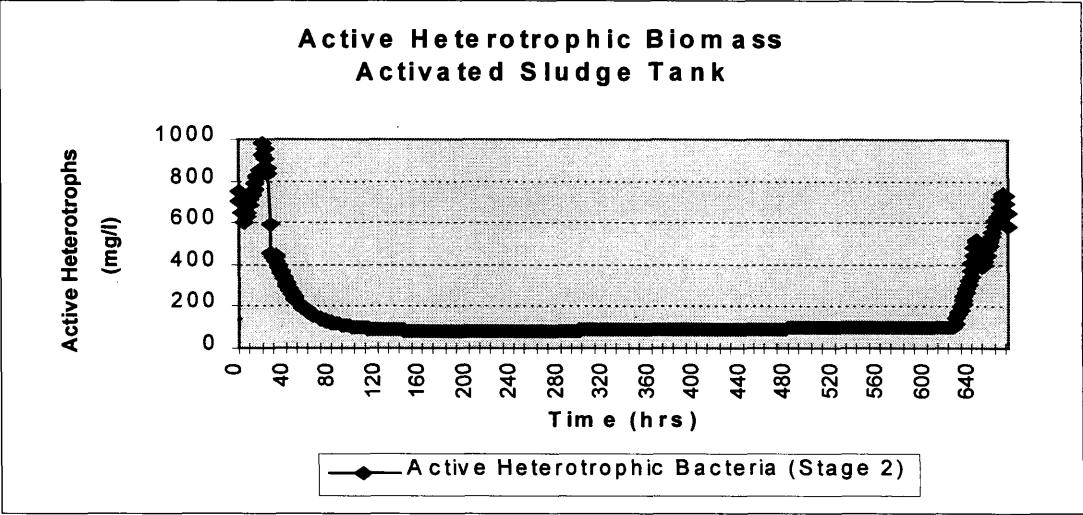


Fig. E.57 WTP Effluent (DWF) - Flow

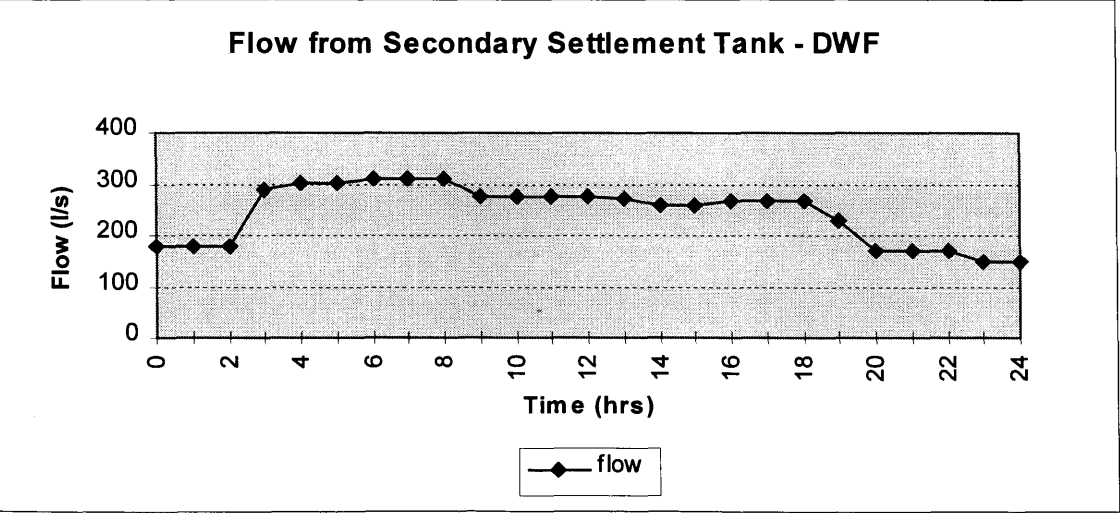


Fig. E.58 WTP Effluent (DWF) - Ammonia

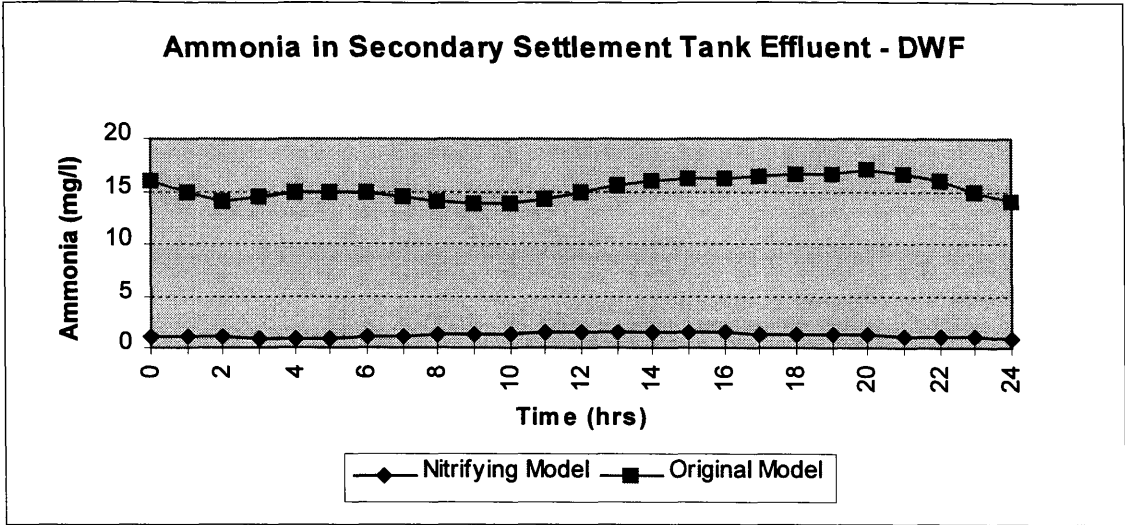


Fig. E.59 Influent of 600l/s for 4days – WTP Effluent - Flow

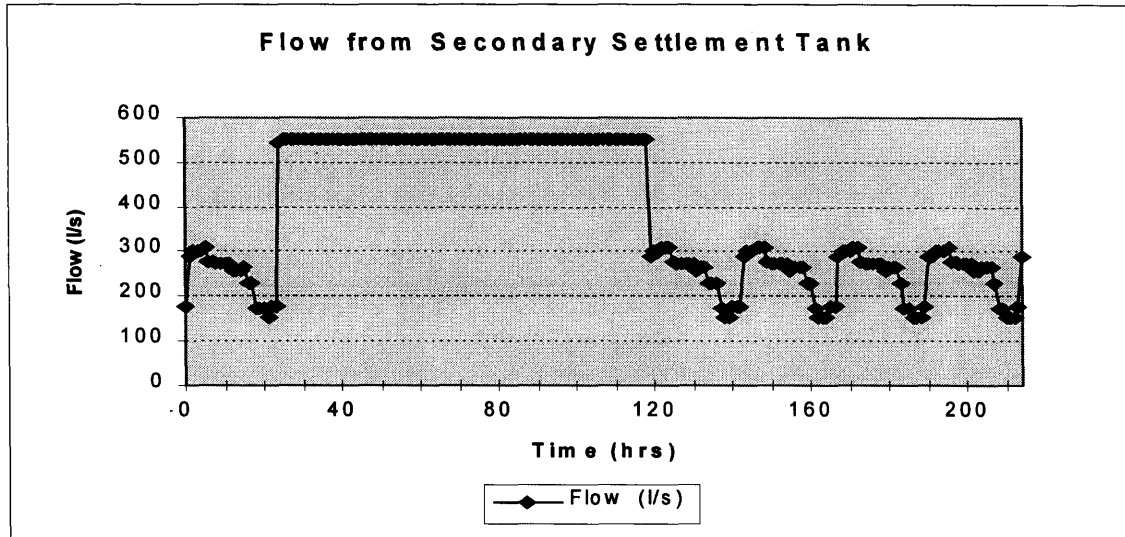


Fig. E.60 Influent of 600l/s for 4days – WTP Effluent - Ammonia

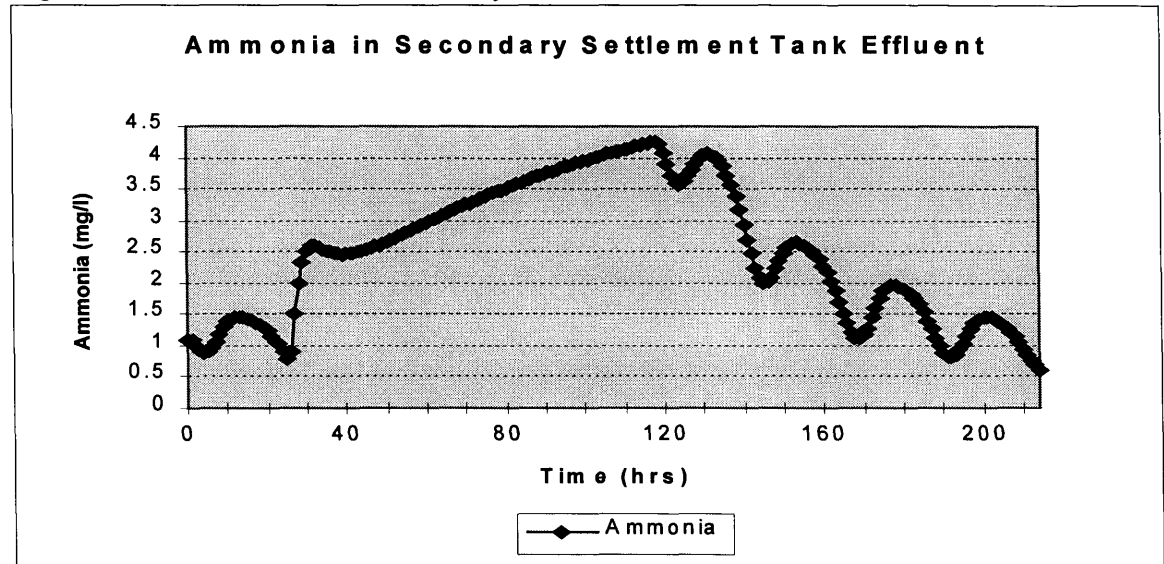


Fig.E.61 Influent of 600l/s for 4 days – Autotrophic Biomass in Activated Sludge Tank

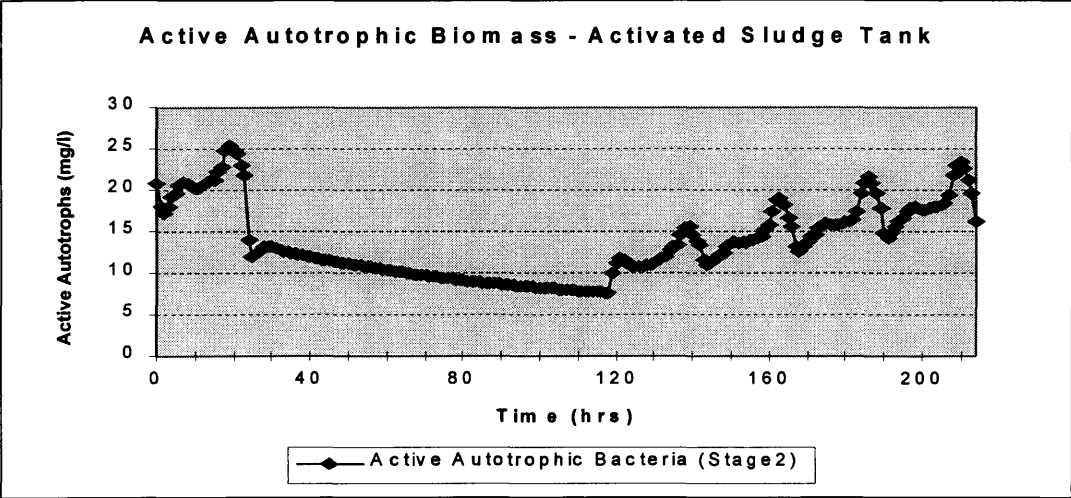


Fig. E.62 Performance Comparison (600l/s and 700l/s for 4 days) – WTP Effluent - Ammonia

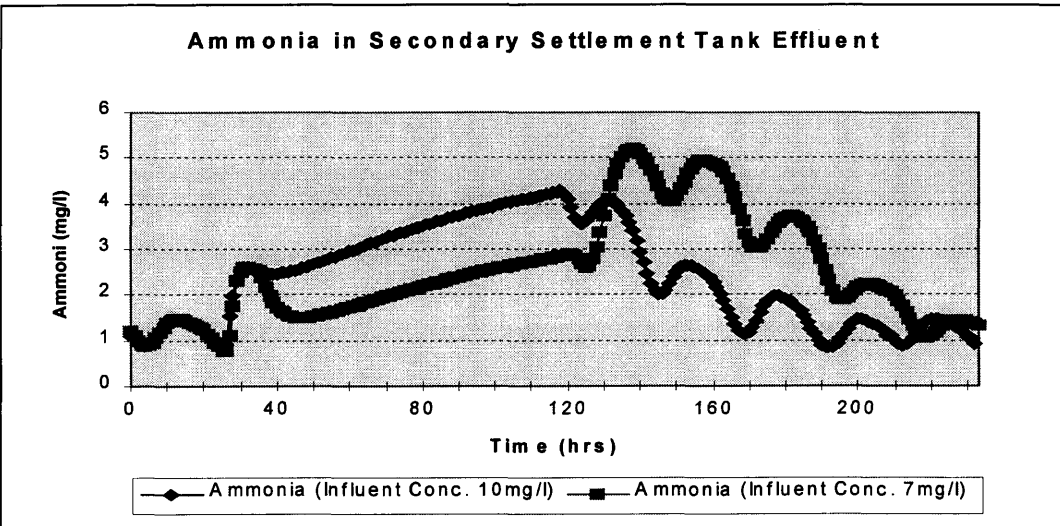
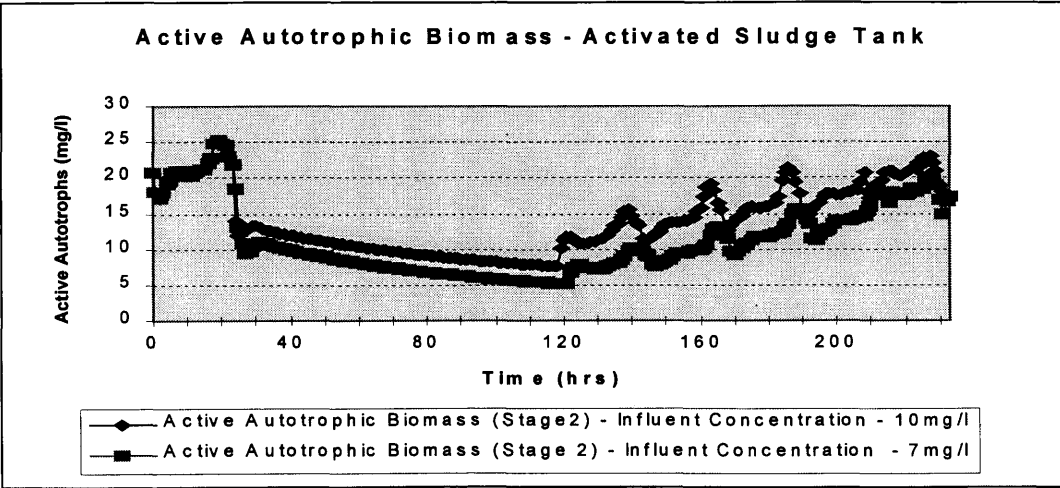


Fig. E.63 Performance Comparison (600l/s and 700l/s for 4 days) – Autotrophic Biomass in Activated Sludge Tank



Appendix F

Biological Treatment -
Two or Three Times DWF?

Contents

F.1	Introduction	F3
F.2	BOD:- 2DWF/3DWF Analysis	F3
F.3	Ammonia:- 2DWF/3DWF Analysis	F4
F.4	Conclusions	F7

List of Figures

F.1	WTP Effluent - Flow	F5
F.2	WTP Effluent from Final Settlement Tank - BOD	F5
F.3	WTP Effluent from Final Settlement Tank and By-pass – BOD	F5
F.4	Heterotrophic Biomass in Activated Sludge Tank	F5
F.5	WTP Effluent – Flow	F6
F.6	WTP Effluent from Final Settlement Tank – Ammonia	F6
F.7	WTP Effluent from Final Settlement Tank and By-pass – Ammonia	F6
F.8	Autotrophic Biomass in Activated Sludge Tank	F6

Biological Treatment

2 DWF or 3 DWF?

F.1 Introduction:-

A subsidiary aim of the research project was to analyse whether the provision of biological treatment of up to twice the dry weather flow would be superior to the adopted UK approach of providing treatment of up to three times the dry weather flow. The analysis was carried out as it could not be ascertained why the three dry weather flow multiple is utilised other than for reasons of operational experience (Wotherspoon, 1997)¹. The analysis in chapter 19 indicated that this test would be of interest as the major cause of poor WTP performance was the displacement of the biomass resulting from the increased hydraulic loads. Protecting the reactor via adjustments to the control setting would therefore alleviate this problem. The results from this analysis are shown in figures F.1 to F.8.

F.2 BOD:- 2DWF/3DWF Analysis

With respect to figure F.2 it is observed that BOD effluent from the activated sludge unit is significantly reduced with the provision of biological treatment for flows of up to twice the dry weather flow. The lower concentrations result as a consequence of the greater residence time which exist within the reactor. This allows a greater degree of treatment to occur.

However, with reference to figure F.3, it can be seen that the best overall performance (final effluent) with respect to BOD is obtained when treatment is provided for up to three times the DWF. The reason being that this multiple produces

1. **Wotherspoon, D.J.J.W. (1997)** Verbal Communications Concerning NoSWA WTP Operational Procedures.

lower volumes of settled sewage which bypass the biological treatment process. Consequently, the final effluent, which includes the treated sewage and bypass sewage, is kept at a concentration closer to that of the treated sewage. It is thus concluded that the better treatment multiple for BOD is three times the dry weather flow.

Figure F.1 shows the protection which the two dry weather flow system gives to the active biomass concentrations. However, as no period of continued disruption is experienced for the BOD determined, due to the rapid growth rates of the heterotrophic bacteria, the protection provided by the flow split provides no real benefit in terms of the overall performance).

F.3 Ammonia:- 2DWF/3DWF Analysis

The benefit of protecting the biomass from hydraulic displacement is seen with respect to the nitrification process. This is demonstrated by figs. F.5 through F.8. With reference to fig. F.6, which shows ammonia effluent concentrations from the activated sludge unit, it can be observed that significantly greater reductions in ammonia are obtained when biological treatment of up to twice the dry weather flow is provided. Similar to the previous discussions this is a consequence of the increased residence time within the reactor.

Figure F.8 shows the concentrations of autotrophic biomass. It can be observed that the 2 DWF split does not only help reduce the sludge displacement, but allows a general trend of increasing biomass to occur. This again is a consequence of the increased residence time within the reactor allowing a greater opportunity for biomass growth. It is therefore noted that when the dry weather flows re-establish the biomass are at a level which prevent a period of continued disruption from occurring.

BOD Analysis

Fig. F.1 WTP Effluent - Flow

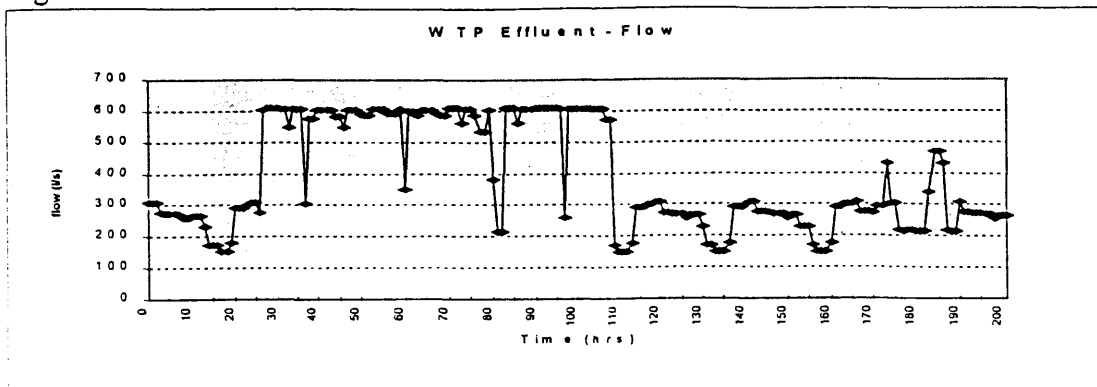


Fig. F.2 WTP Effluent from Final Settlement Tank - BOD

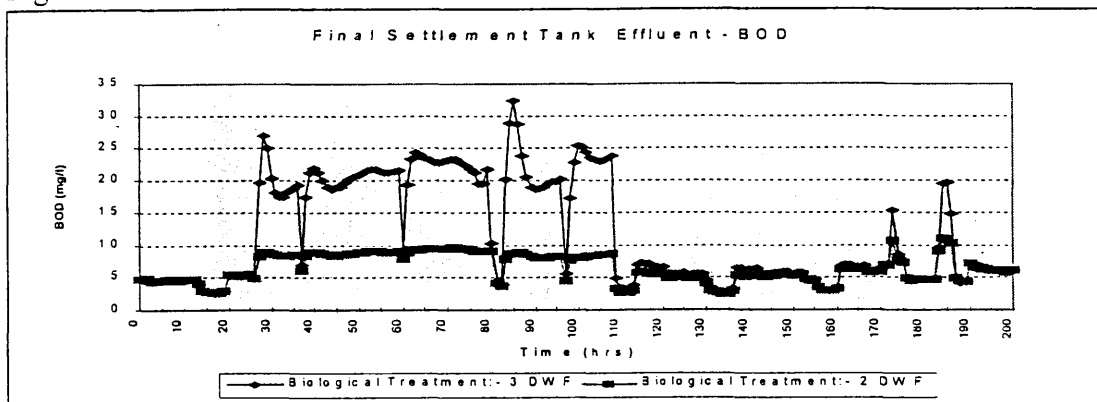


Fig. F.3 WTP Effluent from Final Settlement Tank and By-pass - BOD

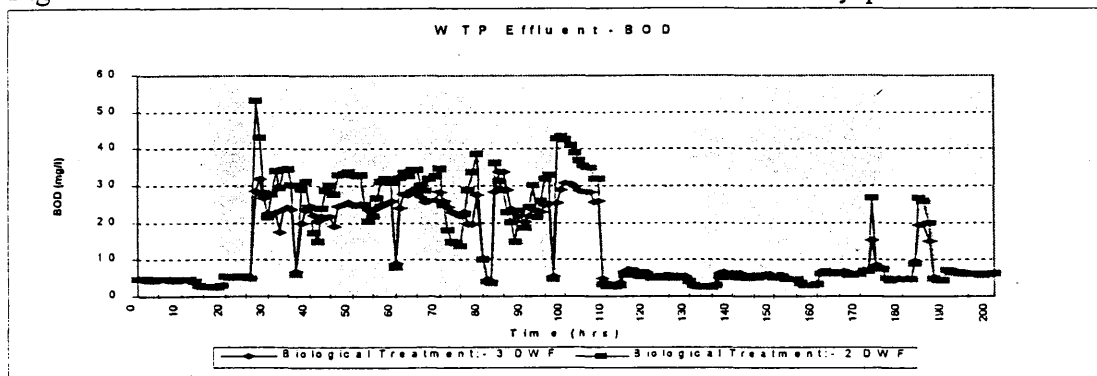
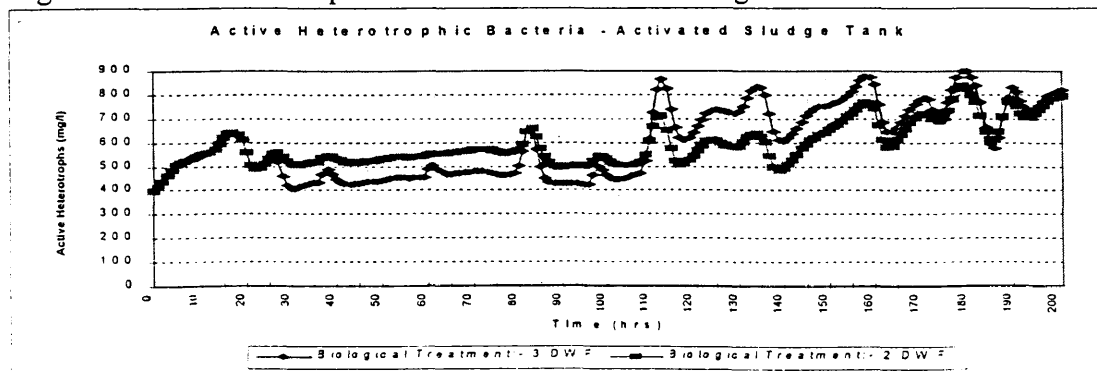


Fig. F.4 Heterotrophic Biomass in Activated Sludge Tank



Ammonia Analysis

Fig. F.5 WTP Effluent - Flow

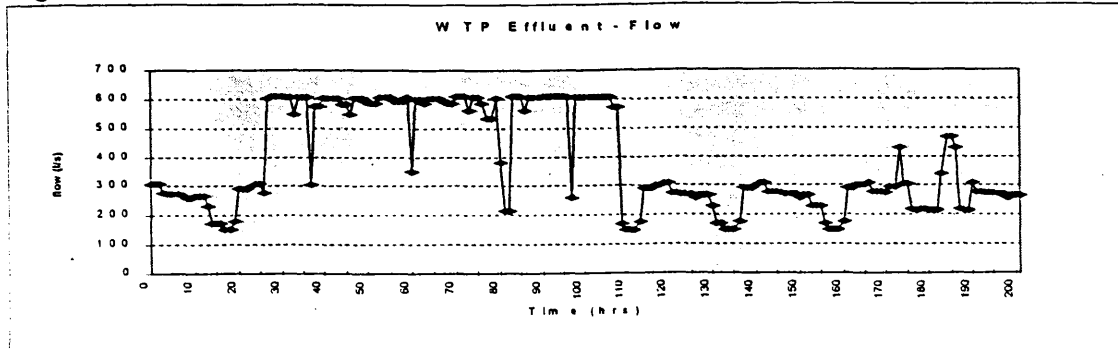


Fig. F.6 WTP Effluent from Final Settlement Tank - Ammonia

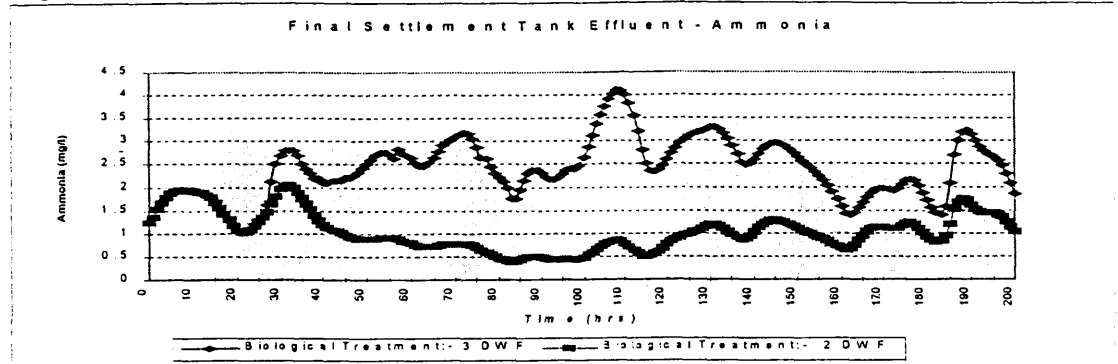


Fig. F.7 WTP Effluent from Final Settlement Tank and By-pass - Ammonia

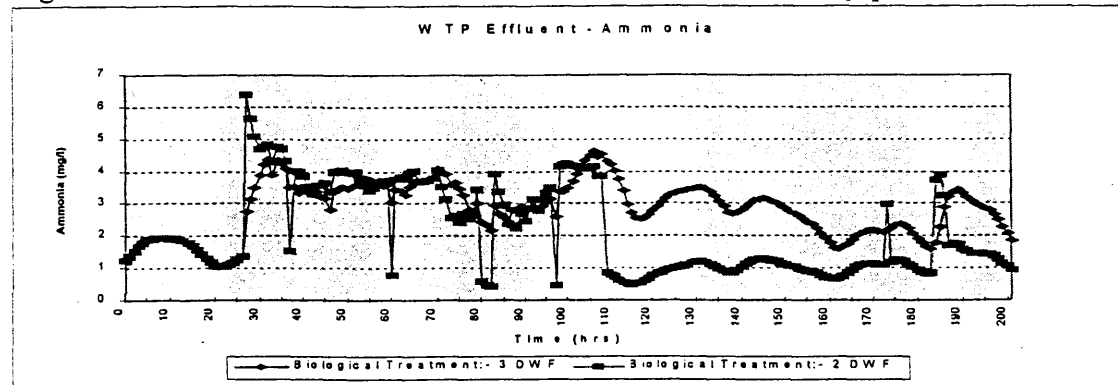
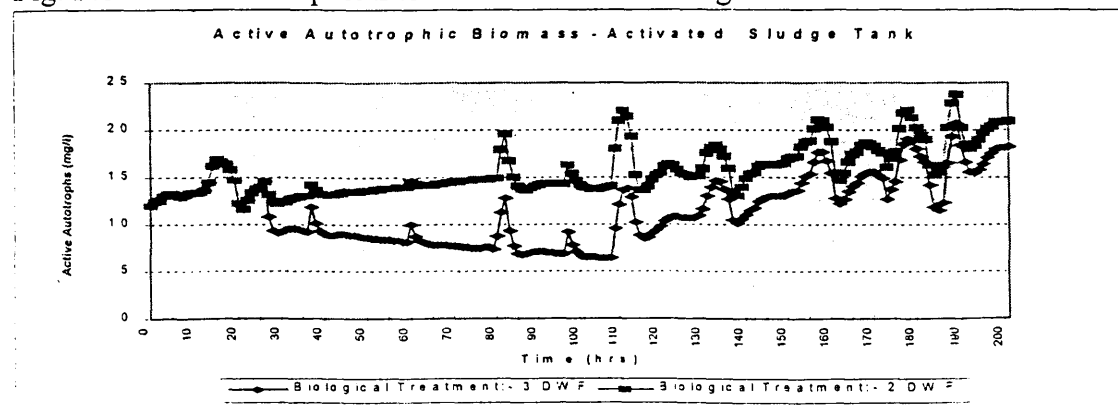


Fig. F.8 Autotrophic Biomass in Activated Sludge Tank



F.4 Conclusions

A significant benefit was obtained with respect to the nitrification process via the utilisation of the two dry weather flow split. Unfortunately this benefit was countered by poorer BOD performance. Consequently, the biological treatment of up to only twice the dry weather flow cannot be considered better or worse than the treatment of three times the dry weather flow, as no overall improvements were observed. This corresponds with the work carried out by **Guderain et al, (1997)**², who concluded that variations in the flow to treatment has different effects upon different determinands.

2. **Guderain, J., Durschlag, A. and Bever, J. (1997)** Total Emissions from Biological Reactors in Wastewater Treatment Plants and Combined Sewer Overflows. The Sewer as a Physical, Chemical and Biological Reactor. 2nd Int. Conf. 25 - 28th May, Aalborg.

Appendix G

Publications

List of Publications

1. Petrie M.M & Jack A.G. (1994)

The Development of an Integrated Catchment Management Plan for the City of Perth. Wallingford Procedure User Group. November Conference, Blackpool.

2. Jack A.G., Petrie M.M. & Ashley R.A. (1995)

Integrated Catchment Modelling - A Sustainable Approach for the City of Perth?

Third International Conference on Water Pollution:- Modelling, Measuring and Prediction. Water Pollution '95. Porta Carros, Greece.

3. Jack A.G., Petrie M.M. & Ashley R.A. (1995)

The Diversity of Sewer Sediments and the Consequences for Sewer Flow Quality Modelling. International Conference on Sewer Solids - Characteristics, Movement and Control. September 1995. Dundee. Wat. Sci. Tech. Vol. 33 No.9 pp 207-214, 1996.

4. Jack A.G. & Petrie M.M. (1995)

The Capabilities and Limitations of Modelling in Holistic Management Practice.

European Modellers User Group - Blackpool. November Conference.

5. Jack A.G. & Ashley R.A. (1996)

Sewer Flow Quality Modelling and Integrated Management. Seventh International Conference on Urban Storm Drainage. Hannover, Germany. September.

6. Jack A.G, Ashley R.A., Akunna, J., Wotherspoon, D.J.J. & Petrie M.M. (1999)

Total Emission Analysis for Combined Sewers and Wastewater Treatment Plants. 8th International Conference on Urban Storm Drainage, Sydney, Australia.